Chapter 4
Damage Losses Assessment Models

Abstract The chapter focuses on the definition of a loss function which has been one of the first indicators used to determine resilience. Different models to evaluate the damage losses are provided, approaching the problem in probabilistic terms using fragility functions and analyzing the different type of uncertainties which appear in the resilience assessment.

4.1 State of Art on Loss Assessment Models

One of the biggest challenges faced when a seismic event occurs on a structure is to assess the magnitude of the effects caused by a catastrophic event in order to quantify the damages and provide a recovery strategy. Starting from the assumption that it is not possible to assess the aftermath of an earthquake until after it has happened, it may be possible instead to approach the problem in *probabilistic terms* and evaluate the losses through the use of numerical simulations. One of the inherent problems with current structural design practice is that seismic performance is not explicitly quantified. Instead, building codes rely on prescriptive criteria and overly simplified methods of analysis and design that result in an inconsistent level of performance. One way of quantifying earthquake performance that has been proposed by research Aslani and Miranda (2005), is the use of economic losses as a metric to gauge how well structural systems respond when subjected to seismic ground motions. While society and building owners main concern is the protection of life, there are other risks that have traditionally been ignored in earthquake-resistant design, such as the control of economic losses or the definition of an acceptable level of probability that a structure could maintain its functionality after an earthquake. Advancements in Performance-Based Earthquake Engineering (PBEE) methods have demonstrated the need for better quantitative measures of structural performance during seismic ground motions and have improved methodologies for estimating seismic performance. In order to provide a comprehensive discussion about this topic, it is convenient to follow the path taken by The Pacific Earthquake Engineering Research (PEER) Center, who has conducted a significant amount of research to address this need, by formulating a framework that quantifies performance in metrics that are more relevant to stakeholders, namely, deaths (loss

G.P. Cimellaro, *Urban Resilience for Emergency Response and Recovery*,
Geotechnical, Geological and Earthquake Engineering 41,
DOI 10.1007/978-3-319-30656-8_4

of life), dollars (economic losses) and downtime (temporary loss of use of the facility). The PEER methodology uses a probabilistic approach to estimate damage and the corresponding loss based on the seismic hazard and the structural response. The facility performance levels can be expressed qualitatively or quantitatively as shown in Sect. 4.7. *Qualitative performance levels* are the current state of practice and are related to the structural characteristics of the elements based on engineering ad-hoc judgments. On the other hand, *quantitative performance levels* permit to rigorously relate the performance levels of a facility to the structural characteristics of the facility. Economic losses in a facility due to earthquakes could represent a qualitative measure of seismic performance. It is useful to focus the attention on a classification of the economic losses due to a seismic event, assuming that the economic losses in a facility can be categorized as direct and indirect losses. Direct losses are those closely associated with repair or replacement costs of building components, whereas indirect losses are those resulting from the temporary loss of function (downtime) of the facility. In order to express a correct prediction of economic losses that may occur due to earthquake ground motions, it is first necessary to make an accurate prediction of the response of the structure when subjected to earthquake ground motions of different levels of intensity. One of the possible solutions to reach the objective is to use a Probabilistic Seismic Hazard Analysis (PSHA): a rational procedure through which it is possible to estimate the annual probability of exceedance of spectral ordinates at a given site by taking into account the location and seismicity of all possible seismic sources that can affect the site. The next step would be a Probabilistic Seismic Structural Response Analysis (PSSRA), which extends a PSHA to the estimation of the annual probability of exceedance of the Engineering Demand Parameter (EDP). The use of response history analyses applying accelerograms scaled at various levels of intensity to investigate the response of structures at various levels of ground motion intensity has been also referred to as “dynamic pushover analysis” (Luco and Cornell 1998) or “incremental dynamic analysis” (Vamvatsikos and Cornell 2001).

4.2 Regional Seismic Losses Assessment Models (RSLA) for Ordinary Buildings

Most of the research related to PBEE has focused on quantifying the possible risks to individual buildings. However, parties interested in a group of geographically distributed buildings, such as policy makers, insurers, practicing engineers working in city planning and real-estate developers, need to make risk-informed decisions on a regional or portfolio, rather than an individual building, basis (Liel and Deierlein 2012). For this reason, over the past decade, researchers have developed methods to extend PBEE to assess the risk of earthquake-induced losses for groups of buildings (hereafter referred to as “Regional Seismic Loss Assessment of buildings (RSLA)”). These methods predict the expected loss, as well as the variation therein, recognizing that risk-informed decision-making depends upon quantifying the likelihood of experiencing rare, but catastrophic levels of loss (Haimes 1998).

There are many sources of uncertainty affecting the prediction of earthquake-induced regional losses, such as those associated with the characteristics of earthquakes, the properties of ground shaking at different sites, the building response and capacity, the fragility of building components, the costs of repairing damage, etc.

Probabilistic regional seismic loss assessment method rely on Monte Carlo simulation because of the elevated number of uncertainties and the lack of closed-form solutions available to propagate them through the loss assessment process. This amounts to repeating the loss assessment for different sets of probabilistically characterized input random variables to develop a suite of "regional loss realizations" from which statistics for the mean and variance in regional loss can be obtained.

Regional loss studies aim at the estimation of economic losses for a large number of buildings, while on the other hand the Building-specific loss estimation studies aim at providing more accurate estimations of economic losses for specific buildings located at specific sites. Regional methods do not provide the necessary level of detail required by performance-based earthquake engineering (Aslani and Miranda 2005), but they can be used when Resilience analyses want to be taken at the regional level. Regional loss estimations attempt to quantify losses for a large number of buildings within a specific geographic area. One of the first investigations to explicitly consider the probabilistic nature of seismic-induced monetary losses was the study by Whitman et al. (1973), which introduced the concept of damage probability matrices into loss estimation methodology, where damage ratios were used to describe the amount of estimated damage, and seismic intensity was expressed as a function of Modified Mercalli Intensity (MMI), which was the selected ground motion intensity measure. In 1992, the Federal Emergency Management Agency (FEMA) and the National Institute of Building Sciences (NIBS) began funding the development of a geographic information system (GIS)-based on regional loss estimation methodology which eventually was implemented in the widely-used computer tool (Hazus 2014). HAZUS is a natural hazard loss estimation methodology implemented through PC-based Geographic Information System (GIS) software developed under agreements with the National Institute of Building Sciences (NIBS). HAZUS as well as the more recent regional loss estimation studies have been conducted developing empirical fragility functions for different classes of building constructions, different typologies of buildings and for several time- and frequency-dependent ground motion parameters. More details about the software can be found in Sect. 14.4.1.

4.3 Seismic Loss Assessment for Infrastructure Systems

In the past decades, research has been focusing on the response of a complex urban lifeline system under external perturbations. Some approaches adopt *analytical system reliability frameworks* to estimate the probabilities of complex system events (Dotson and Gobien 1979; Kang et al. 2008; Li and He 2002; Song and Der Kiureghian 2003) while others rely on *simulation models* to estimate seismic performance of a lifeline system (Hwang and Shinozuka 1998; Shinozuka et al. 2007; Werner et al. 2000). These approaches generally incorporate the vulnerability of components represented by fragility curves in a system level analysis.

Analytical system reliability approaches evaluate statistical measures, such as the probability of system events (e.g., availability of a path from a node to another node) and their associated cut-sets. These approaches are flexible and applicable to generic networks, but they are not applicable to large networks, because the number of system events related to computation is an exponential function of the size of networks, and additional measures beyond statistical ones (e.g., imbalance between supply and demand in power grid and drivers' delay in transportation networks) are required to predict the functional loss of a system (Li and He 2002; Hwang and Shinozuka 1998). On the other hand, *simulation-based approaches* generally use system-specific flow analysis algorithms to compute properties of interests in a system that cannot be obtained from system reliability analysis. Although the simulation-based approaches can require a large number of simulations to achieve acceptable accuracy, and computer run time can be excessive, the obtained properties provide important information to social scientists for quantifying socioeconomic impacts, which is beneficial in comprehensive pre-disaster planning and consequence estimation.

4.4 Loss Function as Resilience Indicator

In statistics a loss function represents the a measure of the degree of inexactness (generally the difference between the estimated value and the true or desired value). Loss estimation has to be defined using damage descriptors that can be easily translated in monetary terms and a series of parameter units that can be measured or counted (e.g. the number of bridges available in a network, or the total length of viable roads). The loss estimation procedure is by itself a source of uncertainty and therefore the problem has to be taken into account in probabilistic terms. In fact, earthquake losses are by nature highly uncertain, and assume a different value for every scenario considered. Despite this, it is still possible to identify some common parameters affecting those losses.

The loss function L_{I,T_RE} can be defined in general as a function of earthquake intensity I and recovery time T_{RE}. The total losses can be divided into two types: Structural losses (L_S), which occur "instantaneously" during the disaster, and Non-Structural losses (L_{NS}), which also have temporal dependencies.

$$L(l, T_{RE}) = L_S(l) + L_{NS}(l, T_{RE}) \tag{4.1}$$

For simplicity, and L_{NS} are described in such a way that it is possible to express the physical structural losses as ratios of building repair and replacement costs using the following relation

$$L_s(I) = \sum_{j=1}^{n}\left[\frac{C_{S,j}}{I_S}\cdot\prod_{i=1}^{T_i}\frac{(1+\delta_i)}{(1+r_i)}\right]\cdot P_j\left\{\bigcup_{i=1}^{n}(R_i \geqslant r_{\lim i})/I\right\} \tag{4.2}$$

where P_j is the probability of exceeding a performance limit state j if an extreme event of intensity I occurs, I is also known as the fragility function; $C_{s,j}$ are the building repair costs associated with a j damage state; I_s are the replacement building costs; r is the annual discount rate: t_i is the time range in years between the initial investments and the time of occurrence of the extreme event; δ_i is the annual depreciation rate. The description of the different methodologies to build fragility curves using performance limit states which might be also uncertain is given in Sect. 4.7.2. Equation (4.2) assumes that the initial value of the building is affected by the discount rate, but the value also decreases with time according to the depreciation rate δ_i, which may vary with time. The nonstructural losses L_{NS} consist of four contributions:

1. Direct economic losses $L_{NS,DE}$ (or Contents losses);
2. Direct Causalities losses $L_{NS,DC}$;
3. Indirect economic losses $L_{NS,IE}$ (or Business interruption losses);
4. Indirect Causalities losses $L_{NS,IC}$;

They are all function of the recovery period. Nonstructural direct economic losses $L_{NS,DE(I)}$ are obtained for every non-structural component k used in the affected system via a formulation similar to Eq. (4.1). The total non-structural direct economic losses are obtained using a weighted average expression as

$$L_{DE}(I) = \left(\sum_{k=1}^{N} w_k \cdot L_{NS,DE,k}(I)\right)\Big/ N_{NS} \tag{4.3}$$

where $L_{NS,DE,k}(I)$ is the non-structural direct economic losses associated with the component k, N_{NS} is the total number of non-structural components in the buildings and w_k is an importance weight factor associated with each non-structural component in the building, that have to be considered in the general model. Direct causalities losses $L_{NS,DC}$ are measured as a ratio of the number of damaged items N_in and the total number of items presented before the event N_{tot}. In the particular case of loss estimation for a building, it can be the ratio between the number N_{in} of injured people (including deaths) over the total number N_{tot} of people in the building.

$$L_{NS,DC}(l) = \frac{N_{in}}{N_{tot}} \tag{4.4}$$

The *indirect economic losses* $L_{NS,IE}(l, T_{RE})$ are time dependent. Because of the different forms these losses can take, they are the most difficult to quantify among the post-earthquake losses. For example, they mainly consist of business interruptions, relocation expenses, rental income losses, etc. Losses of revenue, both permanent and temporary, can be the consequence of damage suffered by structures and contents; this aspect is fundamental and must be taken into account during evaluations of lifelines. A good example may be the structural damage due to the collapse of a bridge span in a major highway. This event generates direct losses, as well as indirect losses subsequent to the loss of revenues as a consequence of impact on the traffic to businesses served. In other cases, there may be some indirect losses due to the disruption that can be more significant than the direct losses. Starting from these considerations, a model is evaluated in which losses due to business interruption should be modeled considering both the structural losses, and the time necessary to repair the structure T_{RE} (Scott and Stephanie 2006). Those quantities are related because the recovery time T_{RE} increases with the extent of structural damage $L_S(I)$. In addition, indirect causalities losses L_{IC} belong to the group in Eq. (4.4).

So, in summary the total non-structural losses L_{NS} can be expressed as a combination of the total direct losses $L_{NS,D}$ and the total indirect losses $L_{NS,I}$.

Direct losses $L_{NS,D}$ and indirect losses $L_{NS,I}$ are also expressed as combination of economic ($L_{NS,IE}$, $L_{NS,DE}$) and casualties ($L_{NS,IC}$, $L_{NS,DC}$) losses as follows

$$L_{NS} = (L_{NS,D} + \alpha_1 L_{NS,I}) \tag{4.5}$$

where

$$L_{NS,D} = L_{NS,DE}^{\alpha_{DE}}(1 + \alpha_{DC} L_{NS,DC}) \tag{4.6}$$

$$L_{NS,l} = L_{NS,IE}^{\alpha_{IE}}(1 + \alpha_{lC} L_{NS,lC}) \tag{4.7}$$

and where α_I is the weighting factor related to indirect losses, α_{DE} is a weighting factor related to construction losses in economic terms; α_{IE} is a weighting factor related to general business interruption; α_{DC}, α_{IC} are the weighting factors related to the nature of occupancy (i.e. schools, critical facilities, density of population). These weighting factors are all determined by socio-political criteria (i.e. cost benefit analyses, emergency functions, social factors, etc.). In the end, L_S and L_{NS} are summed together to obtain the total loss function. The loss function can be used as possible resilience indicator that is time dependent, because it includes the immediate losses caused by the disaster and the post disaster losses with change through the time. However to have a complete description of resilience it is necessary to define the restoration process which is described in the next Chap. 5. Before moving in the description of the recovery models it is necessary to define the different methodologies to determine the probability P_j of exceeding a performance limit state.

4.5 State of the Art on Fragility Curves

Awareness of the potential seismic hazard and the corresponding vulnerability of structures affecting urban areas that created serious economic and social impact have been increasing following recent earthquakes. The prediction of structural damage is critical for the evaluation of the economic losses in earthquake regions and it should be estimated with an acceptable degree of credibility, in order to mitigate potential losses dependent on the seismic performances of structures. Performance can be characterized through fragility functions that express the conditional probability that the building or a component is in, or exceeds, a particular damage state. Major efforts were made in the past in defining, evaluating and quantifying the fragility of structures, following different strategies and approaches. Various studies used Monte Carlo simulations to calculate fragility functions related to a specified structural model, such as Hwang and Huo (1994), Fukushima et al. (1996), Shinozuka et al. (2000a), Karim and Yamazaki (2001), and Kafali and Grigoriu (2005). Other studies developed empirical fragility functions using damage records resulting from past earthquakes, such as Basoz and Kiremidjian (1997) and Shinozuka et al. (2000b). Reinhorn et al. (2001) developed an analytical procedure to evaluate fragility of inelastic structures based on spectral response-capacity analysis and a probabilistic estimate of dynamic response. Karim and Yamazaki (2001) developed an analytical approach for constructing fragility curves of piers of bridges, using nonlinear dynamic response of an equivalent single degree of freedom model of the pier obtained by static pushover analysis. Gardoni et al. (2002) developed a methodology to construct probabilistic capacity models of structural components using a Bayesian approach where the originality of their method consisted in adding correction terms that explicitly describe the inherent systematic and random errors to existing deterministic models already available in literature. However, the developed model can be only applicable to structural systems that have geometry and material properties within the range of observations used to assess the model. Ramarmoorthy et al. (2006) developed fragility curves to assess the seismic vulnerability of a generic two stories reinforced concrete frame using a Bayesian methodology that takes into account aleatoric and epistemic uncertainties. Chaudhuri and Hutchinson (2006) presented an analytical evaluation of fragility curves for a range of rigid, sliding dominant equipment mounted on bench surfaces. In their study, the authors included the uncertainties of friction coefficient and supporting characteristics separately. Goulet et al. (2007) presents a state-of-the-art seismic performance assessment where fragility curves are used to evaluate probabilities of component damage. The emphasis of this study is on the estimation of the expected annual losses and the uncertainties involved in the evaluation of this decision variable, while less effort is made in presenting the procedure of evaluating the fragility curves. Porter et al. (2007) summarized six procedures to evaluate experimental fragility functions, which are considered in the development of a standard for (ATC-58), however not all the procedures available in literature are addressed. More recently, Williams et al. (2009) presented a decision-making methodology to evaluate the benefit of seismically retrofitting existing structures, focusing on the effect of loss reduction, investment return period and retrofitting costs on the feasibility of seismic retrofitting and evaluating the probability of failure combining the conditional probability with the probability of occurrence of a seismic event. Although several works are referenced in this introduction, the survey is by no means comprehensive and it is presented here to highlight several distinct techniques. The limit of the approaches above is that they cannot be verified by laboratory testing, because such verifications require multiple physical models, brought to failure, which are prohibitively expensive and of long duration. Recently FEMA (2007) proposed a standard protocol developed for ATC-58 to evaluate performance of structural and nonstructural components that tries to solve the economical constraints of laboratory specimens by using multiple initial assumptions.

4.6 Analytical Formulation

Seismic fragility functions represent the probability that the maximum response $\mathbf{R}(\mathbf{x}, I, t) = \{\mathrm{R}_1, \ldots.., \mathrm{R}_n\}$ of a specific component, structure, or family of structures, exceeds a threshold $\mathbf{R}_{LS}(\mathbf{x}, I) = \{\mathrm{R}_{LS1}, \ldots .\mathrm{R}_{LSn}\}$ associated with a desired limit state, conditional on the *earthquake intensity measure, I.* The response $\mathbf{R}$ and the response thresholds $\mathbf{R}_{LS}$ are functions of the same structural/nonstructural properties of the system $\mathbf{x}$, the ground motion intensity I and the time t. In the formulation, it is assumed that the response threshold $\mathbf{R}_{LS}(\mathbf{x})$ does not depend on the ground motion history, while the ith response $R_i(\mathbf{x}, I, t)$ of any component or structure is represented by its maximum value over the duration of the response history, $R_i(\mathbf{x}, I)$. The detailed description of dependence of the response, $\mathbf{R}(\mathbf{x}, I)$, on $\mathbf{x}$ and I, and the dependence of the response threshold $\mathbf{R}_{LS}(\mathbf{x})$ on $\mathbf{x}$ will be omitted in the following formulation for simplicity of presentation. Additionally, the chapter follows the standard convention of denoting random variables in capital letters and constants in lowercase letters. With these assumptions, the general definition of fragility *FRLS* based on earthquake intensity I can be written as

$$F_{R_{LS}}(i) = P\left(R_i \geq r_{LSi} | I = i\right) \tag{4.8}$$

where $R_i =$ ith random variable of the response that can be either a deformation quantity, such as interstory drifts, or a force quantity, such as bending moment or shear force, or a combination thereof, or any other measure of damage for which adequate capacity models exist (Badillo-Almaraz and Cimellaro 2006); $r_{LS,i} =$ response threshold, or limit state, related to a certain functionality or damage; $I =$ *earthquake intensity* measure, which can be represented by PGA = Peak Ground Acceleration; PGV = Peak Ground Velocity; PVS= Pseudo Velocity Spectrum; MMI = Modified Mercalli Intensity scale etc.; and i = a given earthquake intensity value. Even though the earthquake intensity measures above have mostly been used in seismic fragility analysis, a definition of fragility based on *earthquake hazard* (e.g. return period T_r of a given earthquake event, annual probability of exceedance λ, etc.) can also be valuable because seismic hazard curves or maps are generally represented using the return period of the design earthquake. Therefore, in this chapter seismic fragility curves are developed as a function of the return period by utilizing the probability density function interpolation technique (Cimellaro et al. 2006; Yi et al. 2007). In order to find the expression of fragility curves as function of the earthquake hazard, two assumptions are necessary: (i) the structural responses are log-normally distributed under earthquake ground motions corresponding at the same probability of exceedance in Eq. (4.8); (ii) the *seismic hazard curves* of the structural responses are described by the following expression (Cornell 1996):

$$\lambda = H(r_{LS})_{1yr} = P(R \geq r_{LS})_{1yr} = 1/T_r = K_0 \cdot r_{LS}^{-K_1} \tag{4.9}$$

where λ = average annual frequency of exceedance of a given response threshold; $H(\cdot)$ = seismic hazard curve function; T_r = return period between two exceeded response thresholds, K_0 and K_1 = parameters representing the seismic hazard curve. The estimate of λ is a function of the geometry and material properties of the specific structure and therefore it needs to be estimated for each specific building.

In order to determine the fragility function based on *earthquake hazard H*, it is necessary to determine the probability density function (PDF) of an arbitrary maximum structural response, R_i, at a given annual probability of exceedance. Therefore, it is assumed that the maximum structural response R is log normally distributed and expressed as follows

$$f_R(r) = \begin{cases} \dfrac{1}{r\sigma_{\ln R}\sqrt{2\pi}} \cdot e^{-\frac{(\ln(r) - m_{\ln R})^2}{2\sigma_{\ln R}^2}} & r \geqslant 0 \\ 0 & elsewhere \end{cases} \tag{4.10}$$

where m_{lnR} and σ_{lnR} are the log-mean and the log-standard deviation values, respectively. According to Eq. (4.10), the two seismic hazard curves of the median (m_R) and deviated (σ_R) values of the response R are expressed as follows:

$$\lambda_{m_R} = H(r_{LS})_{1yr} = P(m_R \geq r_{LS})_{1yr} = K_{0,m} \cdot r_{LS}^{-K_{1,m}} \tag{4.11}$$

$$\lambda_{\sigma_R} = H(r_{LS})_{1yr} = P(\sigma_R \geqslant r_{LS})_{1yr} = K_{0,\sigma} \cdot r_{LS}^{-K_{1,m\sigma}} \tag{4.12}$$

where $K_{0,m}$, $K_{1,m}$, $K_{0,\sigma}$ and $K_{1,\sigma}$ = constants of the median m_R and the deviated σ_R values, respectively, which are calculated using linear regressions. The log-mean m_{lnR} and the log-standard deviation σ_{lnR} values are related to the median m_R and the deviated σ_R values through the following expression:

$$m_R = e^{m_{\ln R}}, \ \sigma_R = e^{m_{\ln R} - \sigma_{\ln R}} \tag{4.13}$$

Therefore, the log-mean m_{lnR} and the log-standard deviation σ_{lnR} values at a given annual frequency of exceedance (or return period t_r) are given by the following expression:

$$m_{\ln R}\left(t_r\right) = -\frac{1}{K_{1,m}} \log\left(\frac{1}{t_r \cdot K_{0,m}}\right) \tag{4.14}$$

$$\sigma_{\ln R}\left(t_r\right) = m_{\ln R} + \frac{1}{K_{1,\sigma}} \log\left(\frac{1}{t_r \cdot K_{0,\sigma}}\right) \tag{4.15}$$

Back-substituting Eqs. (4.14) and (4.15) into Eq. (4.10), the PDF of the maximum structural responses corresponding to a given annual frequency λ or return period t_r is obtained as follows:

$$f_R\left(r, t_r\right) = \begin{cases} \dfrac{1}{r\sigma_{\ln R}\left(t_r\right)\sqrt{2\pi}} \cdot e^{-\dfrac{\left(\ln(r) - m_{\ln R}\left(t_r\right)\right)^2}{2\sigma_{\ln R}\left(t_r\right)^2}} & r \geqslant 0 \\ 0 & elsewhere \end{cases} \tag{4.16}$$

Finally, the definition of fragility based on e*arthquake hazard H* is given by the following integral:

$$F_{R_{LS}}\left(t_r\right) = P\left(R_i \geqslant r_{LSi} \middle| T_r = t_r\right) = \int_{r_{LS}}^{\infty} f_R\left(r, t_r\right) dr \tag{4.17}$$

where the hazard is given by the return period, t_r of a given earthquake event. It is important to mention that there is not a one-to-one correspondence between earthquake intensity, I, and earthquake hazard, H as shown in Fig. 4.1.

In fact, different values of earthquake intensities I (PGA, PGV, PVS, S_a and etc.) can correspond to a unique earthquake hazard (e.g. T_r, the annual frequency of exceedance λ etc.).

The advantage of the second formulation in Eq. (4.17), with respect to Eq. (4.8), is that it takes into account directly the uncertainties of occurrence in estimating the earthquake intensity parameters I at the site. Therefore, in professional practice, buildings are designed according to a given return period T_r, that is related to a given earthquake event. It is possible to directly use the expression of fragility function given in Eq. (4.17) for evaluating directly the probability of functionality, or damage, of the system. The details about the method for generating fragility curves according to Eq. (4.17) are given in the following paragraphs. When the number of response parameters to be checked is n, the definition of fragility given in Eq. (4.17) can be written in the following form:

$$F_{R_{LS}}\left(t_r\right) = P\left(\bigcup_{i=1}^{n} \left(R_i \geqslant r_{LSi}\right) \middle| T_r = t_r\right) = \int_{r_{LS}}^{\infty} f_R\left(r, t_r\right) dr \tag{4.18}$$

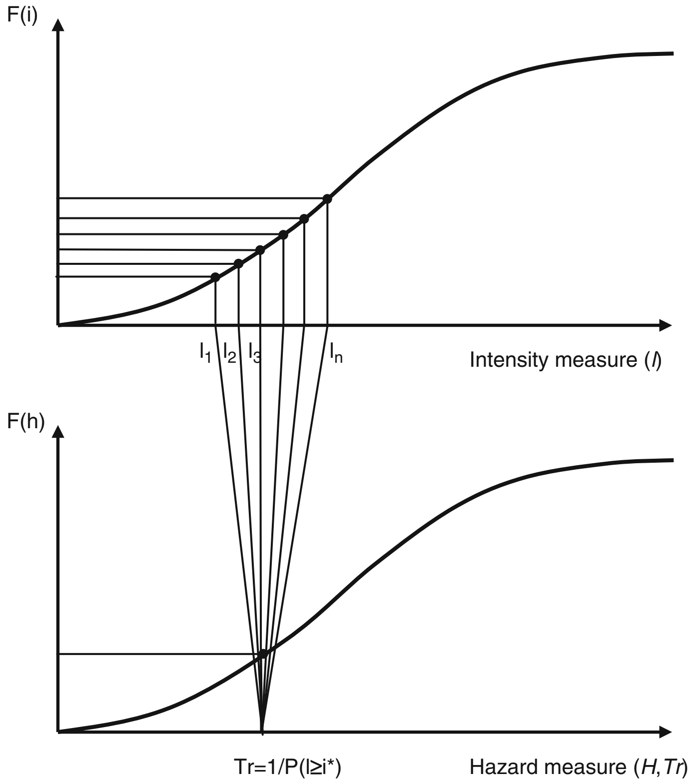

Fig. 4.1 Earthquake intensity versus earthquake hazard fragility curves

where the first right term of Eq. (4.18) is the conditional probability based on the earthquake hazard of the multi-component response exceeding a multidimensional limit state. When the problem is reduced to a bi-dimensional case considering for instance, displacements and accelerations at a specific story of a building, the fragility curve in Eq. (4.18) can be determined using the following expression:

$$F_{\mathbf{R_{LS}}=\begin{bmatrix} D_{LS} \\ A_{LS} \end{bmatrix}}\left(T_r=t_r\right)=\left(P\left((\Delta\geqslant D_{LS})\cup(Z\geqslant A_{LS})|\,T_r=t_r\right)=P\left((\Delta\geqslant D_{LS})|\,T_r=t_r\right)+\right.$$
$$\left.+P\left((Z\geqslant A_{LS})|\,T_r=t_r\right)-P\left((\Delta\geqslant D_{LS})\,(Z\geqslant A_{LS})|\,T_r=t_r\right)\right) \tag{4.19}$$

where Δ = a random variable representing the displacement response, Z = a random variable representing the acceleration response, D_{LS} = displacement threshold, A_{LS} = acceleration threshold and $(\Delta \geq D_{LS})$ and $(Z \geq A_{LS})$ are assumed to be two independent events. The response of the structure can be visually represented for two variables by a "bell surface" (Bruneau and Reinhorn 2007) where the

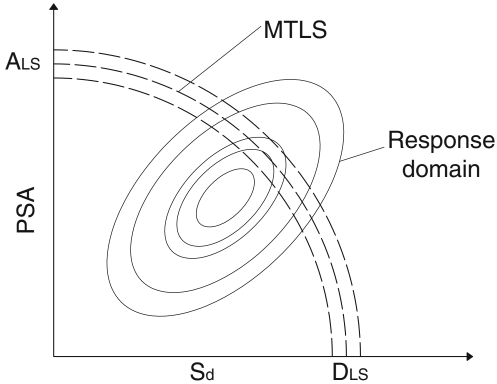

Fig. 4.2 Response domains and the multidimensional threshold limit state

x-axis is the spectral displacement S_d, the y-axis is the pseudo-spectral acceleration, designated here as S_a, and the z-axis is the probability of occurrence (Fig. 4.2). This surface is the joint probability density function of the response expressed in terms of the two variables, the maximum spectral displacement S_d and the maximum spectral acceleration S_a that are assumed to be log-normally distributed.

4.7 Definition of Performance Limit States

4.7.1 Multidimensional Performance Limit State Function

The evaluation of fragility requires a definition of a threshold vector R_{LS}, representing the given limit state. The response vector R and the threshold vector R_{LS} being used in the estimation of fragility must have the same components (e.g. acceleration, drift etc.). Usually, the components of the threshold vector are assumed mutually independent. However for various systems in a structure or substructure, combinations of mutually dependent components, such as accelerations, displacements, drift, velocities, etc., can represent their limits of functionality, or damage. The generalized **M**ultidimensional **T**hreshold **L**imit **S**tate (MTLS) function provides a tool that allows consideration of these dependencies among different components of the threshold vector related to different quantities. The MTLS function $g(R, R_{LS})$ for the case in which n different types of response parameters are considered simultaneously can be defined in n-dimensional form by the mathematical "surface"

$$g\left(\mathbf{R}, \mathbf{R_{LS}}\right) = \sum_{i=1}^{n} \left(\frac{R_i}{R_{LS,i}}\right)^{Ni} - 1 \tag{4.20}$$

where $R_i = i$th component of the response vector (e.g. drifts, accelerations, forces, velocities, etc.); $R_{LS,i} == i$th component of the threshold vector, representing the one-dimensional (1D) limit states and N_i = interaction factors determining the shape of the n-dimensional surface. The limit state defining the boundary between desired and undesired performance corresponds to $g = 0$. When $g < 0$ the structure is safe, while when $g \geq 0$ the structure is not safe (undesired performance). The relation among different thresholds' parameters can be determined through calibration of the MTLS function that is obtained using probabilistic analysis, or engineering judgment based on field reconnaissance data collected after an earthquake or derived from laboratory tests. The model can also be continuously updated as soon as more data are available using the Bayesian approach proposed by Gardoni et al. (2002) and adding correction terms to the proposed limit state function. The MTLS function can be used "locally" to describe the limit state of a single nonstructural component (e.g., scientific equipment, piping and utility systems, etc.) or "globally" to describe the limit state of a part of a sub-structural system (e.g. building story level) or to describe the entire building structure including its nonstructural components. This model can be used to construct fragility curves considering different response parameters (e.g. forces, displacements, velocities, accelerations etc.) combined in a unique fragility formulation. In the proposed formulation, the limit states can be considered either linear or nonlinear dependent, or independent. All these options can be formulated as particular cases of the more general case, with a suitable choice of the parameters involved. In bi-dimensional form, the MTLS function in Eq. (4.18) can be seen in Fig. 4.2 and expressed by the following equation:

$$g\left(\mathbf{R},\mathbf{R}_{\mathbf{LS}}\right) = \left(\frac{A}{A_{LS}}\right)^{N_a} + \left(\frac{D}{D_{LS}}\right)^{N_b} - 1 \tag{4.21}$$

where A_{LS} and D_{LS} = acceleration and drift limit thresholds, respectively; A and D = peak acceleration and displacement response, respectively; N_a and N_b = coefficients determining the shape of the limit state surface. The thresholds A_{LS}, D_{LS} and the coefficients N_a and N_b are determined from either (1) field investigations after an earthquake or from (2) laboratory experiments. The first procedure implies collecting past earthquake field data (Shinozuka et al. 2000a,b). Damage data are related to drifts and they can be determined by field observations, while acceleration thresholds can be determined in the field only when the building is monitored with accelerometers. However, other types of damage data can be determined from laboratory experiments (e.g. number of tiles that fell out of a suspended ceiling) (Badillo-Almaraz and Cimellaro 2006; Retamales et al. 2006). The advantage of the latter procedure is that a range of earthquake intensities can be applied in a controlled fashion to the structure of interest, and inter-story drifts, accelerations, or other parameters, can be monitored and measured more accurately than in the field. However, both methods require multiple outcomes (e.g. structural collapses), which are prohibitively expensive in costs (in laboratory experimental tests) and human lives (in real earthquakes). Therefore, such limit thresholds would have to be derived by numerical analyses using basic engineering principles and rules of mechanics. When the MTLS function is calibrated, A_{LS} and D_{LS} can be assumed as either random variables, or deterministic quantities, either dependent or independent.

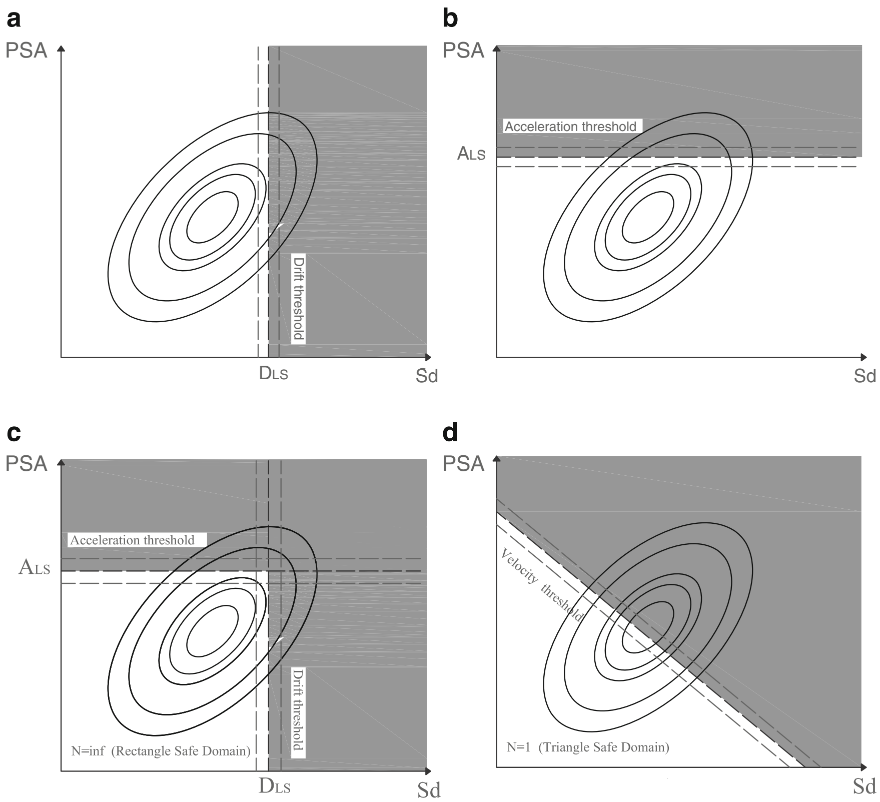

Fig. 4.3 Threshold limit states: (**a**) drift threshold limit state; (**b**) acceleration threshold limit case; (**c**) independent acceleration and inter-story drift limit states; (**d**) velocity limit state

All cases can be considered as particular realizations of the general Eq. (4.18). The two-dimensional (2D) MTLS function in Eq. (4.19) is considered for illustrative purposes. For example, the most common and simplest form of performance function considers only the drift as one-dimensional threshold and it can be obtained assuming $A_{LS} = \infty$; therefore Eq. (4.19) becomes

$$g\left(\mathbf{R}, \mathbf{R_{LS}}\right) = \left(\frac{D}{D_{LS}}\right)^{N_b} - 1 \tag{4.22}$$

where D = displacement response; D_{LS} = displacement threshold that can be either a deterministic or a random variable (Fig. 4.3). In order to be safe, $g \leq 0$ implies $D \leq D_{LS}$. Alternatively, if acceleration limit state is given, then this can be determined assuming $D_{LS} = \infty$, therefore Eq. (4.19) becomes

$$g\left(\mathbf{R}, \mathbf{R_{LS}}\right) = \left(\frac{A}{A_{LS}}\right)^{N_a} - 1 \tag{4.23}$$

As shown in Fig. 4.3b where A = acceleration response; A_{LS} = acceleration threshold that can be either a deterministic or a random variable. In order to be safe, $g \leq 0$ implies $A \leq A_{LS}$. The shape of the performance function is useful for nonstructural components such as acceleration sensitive equipment (i.e. computers, electric devices, lab equipment, etc.). Damage to this type of nonstructural components has gained significant attention following recent earthquakes, because in essential facilities like hospitals, failure of such equipment may hinder emergency response immediately after an earthquake. Most of these components are short and rigid and are dominated by a sliding-dominated response (Chaudhuri and Hutchinson 2006). The case in which both accelerations and inter-story drifts thresholds are considered as independent limit states can be determined from the generalized MTLS function in Eq. (4.19) by imposing $N = N_a/N_b = \infty$ (Fig. 4.3c)

$$g\left(\mathbf{R}, \mathbf{R_{LS}}\right) = \left(\frac{A}{A_{LS}}\right)^{\infty} + \left(\frac{D}{D_{LS}}\right) - 1 \tag{4.24}$$

In fact, if $A/A_{LS} \leq 1$ then ; therefore Eq. (4.24) reduces to

$$g\left(\mathbf{R}, \mathbf{R_{LS}}\right) = \frac{D}{D_{LS}} - 1 \tag{4.25}$$

that corresponds to the inter-story drift limit state given in Eq. (4.24). By imposing the safety condition ($g \leq 0$) in Eq. (4.24), then, therefore Eq. (4.24) becomes the acceleration limit state given in Eq. (4.23). On the other hand, assuming a linear relationship between acceleration and inter-story drift limit states for $\mathrm{N} = \mathrm{Na}/\mathrm{Nb} = 1$, a velocity limit state is obtained, as shown in Fig. 4.3d.

4.7.2 Uncertainties of Limit States

The PLS represent the level of response for a certain functionality limit, or for a specific damage condition. The limits of functionality or of damage depend on mechanical properties, such as strength and deformability, which are in themselves uncertain and therefore in literature are available Bayesian approaches that properly account for all the most relevant uncertainties (Gardoni et al. 2002). Unfortunately, current engineering practice in developing fragility curves is still based on deterministic PLS, usually obtained from scientific/engineering laws, observational data from laboratory experiments or field investigations, design standards, engineering experience, subjective judgment and etc. The main reason for this choice is justified by the fact that the uncertainty in the earthquake load is considerably larger than the uncertainty in the PLSs themselves. In this section, PLSs are considered as random variables, and are defined in terms of both inter-story drifts and accelerations, since the functionality and the failure modes in the case study (presented later) are governed by both. Several cases are considered for the estimation of the fragility related to specific PLS, assuming that the limit thresholds are random variables. For simplicity of explanation, only two response parameters are considered: inter-story drift and floor acceleration. It is assumed that the peak responses have a lognormal distribution, which is used to model positive random variables, following Clough and Penzien (1993). Additionally, Cimellaro et al. (2009) have verified the assumption comparing different PDF (normal, Gumbel, and log- normal) for different structural configurations and results showed that the lognormal distribution is the best fit for the response distribution. For each case, different assumptions are made regarding the random variables considered. An analytical solution is formulated to calculate the probability of exceeding a certain performance limit state, given the probability distribution function of the response and of the limit states. The simplest case is where the inter-story drift threshold d is considered as a deterministic quantity, and is compared with the random variable δŤ of the inter-story drift response that takes only positive values and is assumed to be log-normally distributed as follows

$$f_{\Delta}(\delta) = \begin{cases} \dfrac{1}{\delta\sigma_{\Delta}\sqrt{2\pi}} \cdot e^{-\dfrac{(\ln(\delta) - m_{\Delta})^2}{2\sigma_{\Delta}{}^2}} & \delta \geqslant 0 \\ 0 & elsewhere \end{cases} \tag{4.26}$$

In this case the fragility function describing the probability of exceeding the given performance limit state d is

$$F_d\left(t_r\right) = P\left(\Delta \geqslant d \middle| T_r = t_r\right) = 1 - F_{\Delta}(d) = 1 - \int_0^d f_{\Delta}(\delta)d\delta \tag{4.27}$$

Instead, if the inter-story drift threshold is also a random variable that takes only positive values (but it is assumed independent from Δ), then the fragility function describing the probability of exceeding the given performance limit state D in Eq. (4.27) becomes

$$F_D\left(t_r\right) = P\left(\Delta \geqslant D \middle| T_r = t_r\right) = P(\underbrace{\frac{\Delta}{D}}_{Y} \geqslant 1) = P(Y \geqslant 1) = 1 - F_Y(1)$$

$$= 1 - \int_0^{\infty} f_D(u)F_{\Delta}(1 \cdot u)du; \tag{4.28}$$

where

$$F_Y(y) = \int_0^{\infty}\int_0^{y\cdot u} f_\Delta(\delta) f_D(u) d\delta du = \int_0^{\infty} f_D(u) \int_0^{y\cdot u} f_\Delta(\delta) d\delta du$$
$$= \int_0^{\infty} f_D(u) F_\Delta(y \cdot u) du; \tag{4.29}$$

where u and δ = auxiliary variables; y = specific real number of a real value random variable Y. If both the drift response Δ and the acceleration response Z are considered in the formulation, then these two variables are assumed dependent random variables because generally for every type of structure, experimental observations seem to confirm this assumption. For example for linear SDOF systems, the following relation holds

$$\frac{Z}{\omega^2} = \Delta \tag{4.30}$$

where $\omega = 2\pi/T$ is the circular frequency of a SDOF of period T. Based on this relationship, the problem is reduced from two-dimensional to one-dimensional:

$$F_{R_{LS}}(t_r) = P(\Delta \geqslant d \cup Z \geqslant a|\, T_r = t_r) = P(\Delta \geqslant d \cup \Delta\omega^2 \geqslant a|T_r = t_r) =$$
$$= P(\Delta \geqslant \min(d, a/\omega^2)\,|T_r = t_r) = 1 - F_\Delta(\min(d, a/\omega^2)) =$$
$$= 1 - \int_0^{\min(d,a/\omega^2)} f_\Delta(\delta)\, d\delta \tag{4.31}$$

The probability of exceeding the limit state can be evaluated when the probability density functions of inter-story drift $f_\Delta(\delta)$ is known. Both parameters of the density functions can be calculated using the maximum likelihood method. While in the two cases shown above, the inter-story drift performance limit state d and the acceleration performance limit state a were considered deterministic quantities, they can also be assumed as random variables log-normally distributed as discussed previously. It is assumed that the acceleration response Z and the displacement response Δ are related in the elastic range, so that the relation $\Delta = Z/\omega^2$ holds, while the performance limit state of inter-story drift D and acceleration A are assumed independent random variables. This assumption is reasonable because, nonstructural components such as electronic devices (e.g. computers, etc.) for example, that are acceleration sensitive, cannot be related to the building PLSs that are typically displacement sensitive. In order to simplify the formulation, two new non-dimensional random variables $X = \Delta/D$ and $Y = Z/A$, are assumed.

In this case, the probability of exceeding the given performance limits state can be expressed as

$$F_{R_{LS}}\left(t_r\right) = P\left(\Delta \geqslant D \cup Z \geqslant A \middle| T_r = t_r\right) = P\left(\underbrace{\frac{\Delta}{D}}_{X} \geqslant 1 \cup \underbrace{\frac{Z}{A}}_{Y} \geqslant 1 \left| T_r = t_r\right.\right) =$$

$$= P\left(\underbrace{\frac{\Delta}{D}}_{X} \geqslant 1 \cup \frac{\Delta}{\frac{A}{\omega^2}} \geqslant 1 \left| T_r = t_r\right.\right) = P\left(\underbrace{\frac{\Delta}{D}}_{X} \geqslant 1 \cup \left(\frac{\omega^2 D}{A}\right) \cdot \underbrace{\frac{\Delta}{D}}_{X} \geqslant 1 \left| T_r = t_r\right.\right) =$$

$$= P\left(\underbrace{\frac{\Delta}{D}}_{X} \geqslant 1 \cup \underbrace{\frac{\Delta}{D}}_{X} \geqslant \frac{A}{\omega^2 D} \left| T_r = t_r\right.\right) = P\left(X \geqslant \min\left(1, \frac{A}{\omega^2 D}\right) \left| T_r = t_r\right.\right) =$$

$$= 1 - F_X\left(\min\left(1, \frac{A}{\omega^2 D}\right)\right) = 1 - \int_0^{\infty} f_D\left(u\right) F_{\Delta}\left(\min\left(1, \frac{A}{\omega^2 D}\right) u\right) du; \quad (4.32)$$

Hence, for the evaluation of the exceedance probability in this case, only the probability density function of the inter-story drift response and the probability density function of the inter-story drift limit state are required. In case that inter-story drift performance limit state, D, and the acceleration performance limit state, A, are nonlinearly related through Eq. (4.21), then the probability of exceedance the *MTLS function* is obtained by substituting Eq. (4.21) in Eq. (4.32)

$$\begin{aligned} F_{r_{LS}}\left(t_r\right) &= P\left(\Delta \geqslant d \cup Z \geqslant a \left| T_r = t_r\right.\right) = P\left(\Delta \geqslant d \cup \Delta\omega^2 \geqslant a \left| T_r = t_r\right.\right) = \\ &= P\left(\Delta \geqslant \min\left(d, a/\omega^2\right) \left| T_r = t_r\right.\right) = \\ &= 1 - F_{\Delta}\left(\min\left(d, A_{LS}/\omega^2\left(1 - \left(D/D_{LS}\right)^N\right)\right)\right) = \\ &= 1 - \int_0^{\min\left(d, A_{LS}/\omega^2\left(1-\left(D/D_{LS}\right)^N\right)\right)} f_{\Delta}\left(\delta\right) d\delta \end{aligned} \quad (4.33)$$

The advantage of this formulation is that a limit state can be expressed as function of the other components, once the coefficients of the model have been calibrated; in this case, only a single parameter is needed to limit both displacements and accelerations. Once the probability of exceeding the PLS is computed analytically, the procedure can be repeated again for different intensity measures, generating the fragility curves using the procedure described in the next section.

4.8 Generation of Fragility Curves

In this section, the method for generating fragility curves uses the return period as the intensity measure to take into account the ground motion parameter I at the site, it is described in the following steps:

- *Step I*: for a given value of earthquake hazard H (e.g., return period Tr of an earthquake event), consider n synthetic or real earthquake records.
- *Step II*: analyze the structural system under each of the earthquake records generated in Step I that is consistent with the given hazard level. Compute the maximum pseudo spectral acceleration (PSA) and spectral displacement (Sd) response for every structural and nonstructural component at each story level. Note that nonstructural components are assumed rigid and rigidly connected to the structure for the particular case study considered.
- *Step III*: estimate the mean and the standard deviation from the δ response samples of Step II and evaluate the lognormal PDF of δ the response distribution.
- *Step IV*: evaluate the probability of exceeding, using the analytical expressions given in previous paragraphs, for the case when uncertainties in the limit states are taken in account. The limit states considered are partly dictated by structural safety (displacements) and partly dictated by functionality (accelerations) at each floor level and they are defined as "story PLS."
- *Step V*: repeat Steps I to IV for different hazard levels and locate all the points corresponding to the different probability P of exceeding the limit state in the plane of probability of exceeding versus earthquake hazard. The number of hazard levels (represented by the return period of the earthquake event), available for design in the USGS database (Petersen et al. 2014), is usually four, which is equal to the number of points available to determine the fragility curves as function of the return period.
- *Step VI*: fit the points obtained in Step V using the lognormal cumulative distribution function. The fragility function is described by the following equation:

$$F_Y(y) = \Phi\left[\frac{1}{\beta}\ln\left(y/\theta_Y\right)\right] \; y \geqslant 0 \tag{4.34}$$

  where Φ = standardized cumulative normal distribution function; θ_y = median of y; and β = standard deviation of the natural logarithm of y. A straightforward optimization algorithm based on chi-square χ^2 goodness-of-fit test allows the estimation of the optimal parameters of the lognormal distribution (θ_y and β).
- *Step VII*: repeat Steps I to VI for every story level and develop floor fragility curves. Then, the performance level of the most critical story is suggested to represent the global PLS for the structure.

The reason of the choice in *Step VII* is justified by the fact that in reality, most structures are a combination of series and parallel systems (Nowak and Collins 2000): structures may not fail when a single member fails, but they can fail before all members fail. In a complex structural system like a hospital, structural and nonstructural components interact with each other, therefore identifying series and parallel systems may be difficult. However, the problem can be simplified assuming the hospital is a series system that demonstrates a weakest link system, because failure of the system corresponds to failure of the weakest element in the system. The proposed approach with respect to other methods available in literature (e.g., Zion method described by Kennedy and Ravindra (1984)) addresses *PLSs* as functions of combined multiple structural parameters and also allows consideration of dependencies among different limit thresholds and uncertainties in the limit states themselves. Therefore, the proposed approach can be considered as an alternative method for describing the vulnerable behavior of nonstructural components that are sensitive to multiple parameters, like partition walls that are drift sensitive during the earthquake in the initial vibration cycles but become acceleration sensitive as cantilever type structures, when they disconnect from the top boundary. Another example of nonstructural components that are sensitive to both accelerations and drifts are the piping systems. The disadvantage of the proposed method is that is based on nonlinear time history analysis coupled with Monte Carlo simulations that are used to characterize the demands in terms of their joint density function $f_D(d)$ where d =generic demand parameter. Therefore, the proposed approach may be prohibitive for complex structural systems where excessive computational demand is required.

4.9 Concluding Remarks

The chapter describes different types of loss assessment models to evaluate regional seismic losses of both ordinary buildings and infrastructures. Then attention shifts toward the definition of a loss function which has been one of the first indicators used to define disaster resilience. Different models to evaluate the damage losses are provided, approaching the problem in probabilistic terms using fragility functions and analyzing different types of uncertainties which appear in the resilience evaluation such as the uncertainties in the limit states.

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