Seismic Assessment of Structures and Lifelines

M. Fragiadakis[*], D. Vamvatsikos, M.G. Karlaftis, N.D. Lagaros, M. Papadrakakis

School of Civil Engineering, National Technical University of Athens, Greece

ABSTRACT

We discuss the current state-of-the-art on the assessment of systems (structures and lifelines) subjected to seismic loading. Severe earthquakes are random events that may have a devastating outcome. Therefore, the treatment of seismic loading and its effects has many layers due to the random nature of the problem and also its consequences. We attempt to provide a wide approach to the problem, referencing the major contributions in the field and discussing the most suitable methods for tackling the challenges involved. Our study extends from modeling seismic hazard to the modeling and analysis of structures and lifelines. Emphasis is given to probabilistic performance estimation frameworks and the rational treatment of uncertainty, a major element of contemporary earthquake engineering.

1. Introduction

Severe and extensive damage caused by recent earthquakes (e.g. in Japan and New Zealand) once again raised skepticism regarding our preparedness against earthquakes. During past decades, and especially after the 1994 Northridge and the 1995 Kobe earthquakes, significant research effort and resources were devoted to bring forward the need for performance-based concepts and also the development of a more rational consideration of uncertainties during design and assessment. These efforts resulted in considerable improvements of design codes and guidelines bringing novel principles to the earthquake engineering practice in seismic-prone regions. However, recent events have shown the necessity for extending our attention to the response of critical facilities and lifelines, while the concept of system failure needs a more careful look. For example, after the 2011 Tohoku earthquake, the Fukushima reactor suffered no structural damage but the failure of its cooling system caused the failure of the system and the emission of radiation. In lieu of the above, the definition and calculation of small failure probabilities for important structures has to be revisited.

[*] Corresponding author.
E-mail address: mfrag@mail.ntua.gr

Earthquake engineering is a fundamental part of civil engineering that has spurred significant research efforts in diverse scientific fields: structural dynamics, seismology, material science, experimental methods, computational methods, probability and statistics, but also economics, sociology and other non-engineering fields [1]. At the core of these efforts stands the fundamental problem of system assessment under earthquake loading, whereby the performance of civil engineering structures, such as buildings, dams, bridges, power-plants, pipelines, tanks etc is evaluated using analytical and/or computational/numerical methods to estimate the structural response of appropriate models (build by a combination of experimental work and engineering judgment) when subjected to modeled earthquake loads. Following recent developments, it is widely accepted that one should be interested not only in the structural response, but mainly in the structural performance, as quantified by annualized or life-cycle losses, e.g., in casualties, repair/replacement/rehabilitation costs, downtime etc. Moreover, depending on the end-user, it is often not a single structure that has to be evaluated, but an ensemble of independent structures, for example a small neighborhood, or the building portfolio of a company, a municipality, or even an entire city. Furthermore, the assessment may entail a set of functionally-interdependent facilities that constitute a *lifeline*, such as a water/power/gas distribution network, a highway network, or the medical care system, all potentially considered at the scale of an entire city, county or country. It is, therefore, evident that the problem of seismic protection and risk mitigation requires a multidisciplinary approach that extends beyond the expertise of geologists and structural engineers.

In full knowledge of the enormity of the subject, we offer a comprehensive review of recent advances in earthquake engineering, focusing on the current state-of-art in the major components needed for the seismic performance assessment of structures and lifelines. We chose to approach the problem through *seven* main sections: (a) modeling of seismic actions, (b) structural modeling, (c) analysis and performance assessment of structures, (d) probabilistic and assessment, (e) soft-computing, and (f) assessment of lifelines. The paper aims to discuss the available tools and procedures for seismic risk assessment providing a global, multidisciplinary look to the problem.

2. Seismic loading

Due to the highly uncertain nature of earthquakes, the assessment of their magnitude, location and rate of occurrence is of paramount importance in earthquake engineering. The amplitude and the frequency content of seismic ground motions, as recorded at various sites, depends on the amount of seismic energy released during the fault rupture and its attenuation from source to site. Therefore, although the amount of energy released from the source depends on the size of the fault rupture, the properties of the seismic waves, as ultimately felt and recorded in the surface, depend also on the amount of energy dissipated due to anelastic absorption and geometric spreading. Moreover, local parameters such as superficial geology, site topography and the presence of structures, may also significantly affect the properties of the ground motions that are finally recorded at the site of interest. The various parameters that affect seismic ground motions, in general, are grouped into three categories. The first characterizes the source of energy release, the second the path along which the energy propagates and the last is the point of observation. The three categories are thus known as *source*, *path* and *site.*

Magnitude, distance and soil properties are the most critical parameters and usually ground motion prediction equations are limited to them. However, a great number of other factors may also be of significance.

Seismic forces on a structure typically are inertia forces produced by the motion of the ground, or forces produced by the differential movement of the supports. For engineering purposes, and depending on the application, we seek simplified approaches to represent earthquake loading. Such approaches should be suitable to our needs and consistent with the associated uncertainties. Therefore, we merely have to be able to describe the characteristics of the ground motion that are of engineering significance, adopting metrics that can be extracted from the ground motions that reflect primarily: the *amplitude*, the *frequency content* and the *duration*. In seismic design codes and guidelines, earthquake loads are represented by the response spectrum of maximum absolute acceleration. However, the most faithful representation is achieved through the entire ground acceleration time-history. The representation of the seismic loading in the form of acceleration time-histories means that the hazard is defined in terms of all of the characteristics of the ground shaking. In addition to amplitude frequency, the energy and the duration of shaking are also significant and have to be considered [2]. The latter information is lost when seismic loading is considered in the form of an elastic spectrum. Thus, depending on the problem and the analysis method at hand (see Analysis methods section) seismic loading may be defined using either response spectra, or acceleration time-histories. Both options require the definition/knowledge of the seismic hazard , since the seismic loads, in principle, have to be compatible with the hazard conditions of the site.

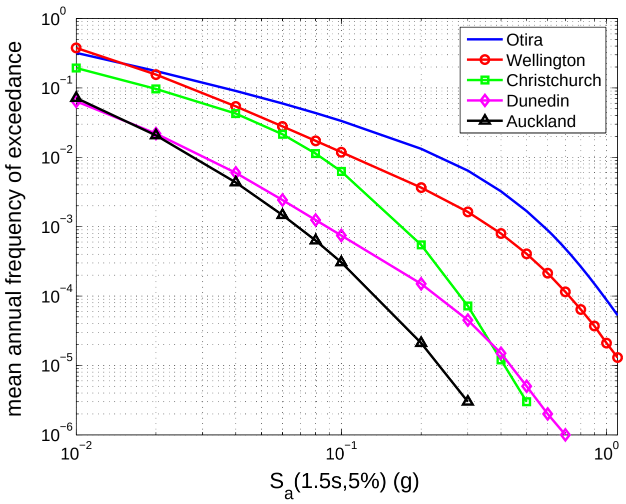


Figure 1. Seismic hazard curves for spectral acceleration for various New Zealand sites (adapted from [3]).

2.1. **Probabilistic seismic hazard analysis**

Probabilistic seismic hazard analysis (PSHA) was introduced by Cornell [4] and Esteva [5] as a comprehensive method to estimate the mean annual frequency (reciprocal of the return period) of seismic events of certain intensity at a given site. At its core, it is akin to a Monte Carlo simulation considering all possible scenarios of earthquake events on the causative sources threatening any site of interest. Such a simulation may be used to produce corresponding scenarios of potential ground motions that threaten the site, in the so-called *non-conditional* risk assessment methods. Typically though, it is employed to estimate the mean annual frequency of exceeding levels of a given (scalar or vector) intensity measure (IM) [6] at the site of interest. Commonly adopted intensity measures are the peak ground acceleration (PGA) and the peak ground velocity (PGV). Spectral values at a given period T, e.g., the 5%-damped, first-mode spectral acceleration, $S_a(T, 5\%)$, are also popular. The intensity measure is a useful concept that serves as an interface variable between seismology and structural engineering, attempting to incorporate all the complexity of the seismic loading into a single quantity that can be used as input for structural analysis, without needing to account for all the diverse characteristics of seismic loading [7]. Hence, the analysis is conditioned on the value of the intensity measure alone, characterizing all *conditional* methods of seismic risk assessment. This effectively separates the work of the seismologist, who only needs to perform PSHA for the intensity measure, from that of the structural engineer, who analyzes the structure considering only the intensity rather than the entire seismological setting. Thus, this approach has achieved wide-spread use in earthquake engineering.

In this traditional role, PSHA adopts a Poisson model of earthquake occurrence to sum up exceedance rates of the IM from all potential scenarios. For each such case, the corresponding IM exceedance frequency is estimating through modulating the baseline seismicity rate by (a) the recurrence relationship (e.g. Gutenberg-Richter) of event magnitudes, (b) the probability function of different possible rupture zones and (c) the probability of exceedance of intensity measure levels for a given event scenario [8]. The latter is essentially the so-called ground motion prediction equation (GMPE), formerly also known as the attenuation relationship. GMPEs are the result of parametric regression on available ground motion recordings to obtain functions for the mean and standard deviation of the seismic intensity given the event magnitude, the source-to-site distance and the soil properties at the site of interest. Convolving all the pertinent probabilistic information is a computationally intensive process that is nearly always handled by specialized (and mainly commercial) software. Relatively recently the OpenSHA initiative [9] has emerged to offer open source PSHA tools, which have now been adapted to form the basis of the OpenQuake Engine [10] of the Global Earthquake Model (GEM) foundation.

The results of PSHA typically appear in the form of seismic hazard curves (Figure 1) for a given (point) site and intensity measure, ground motion fields showing correlated intensities for a given area or uniform hazard spectra, that represent values of spectral acceleration response that have the same return period at the given site across different periods. Such spectra form the basis of current design codes, typically estimated for the 10% probability of exceedance in a 50 years period, or a 475 return period, to provide the design loads for structural analysis for the life-safety limit-state. As discussed by Bommer and Pinho [11], this return period value was initially proposed arbitrarily, but was later found to be sufficient.

2.2. **Ground motion records**

The major difficulty for the analysis of a seismically excited structure is the realistic implementation of seismic actions. It is important that the acceleration time-histories employed in seismic analysis or design are consistent with the seismicity of the region, obtained through PSHA, and representative of the expected or the design earthquake. Especially for reliability assessment using simulation-based methods, a sufficiently large sample size of input ground motions that represent all possible future events at the building site is necessary. In general, this can be a difficult task if accurate representation of all potential ground motion characteristics, as discussed in latter section, is required.

Seismic magnitude affects the frequency content, the duration and the amplitude. Larger events tend to produce longer ground motions that are rich in lower frequencies and have increased amplitude across the board (for a given distance). Near-source sites that lie in the direction of rupture as it travels away from the epicenter are potentially subject to a concentrated wave effect. Ground motion records from such sites are known as "forward-directivity" records. Forward-directivity can produce a prominent acceleration (or velocity pulse), resulting into a narrow-band excitation with concentrated energy at the pulse period. Similarly, the soil profile at the site of interest may selectively amplify or attenuate ground motion frequencies. Soft soil sites often show a resonance effect (most famously in the 1985 Mexico City event) consistent with their eigenperiod that can catastrophically amplify the corresponding frequencies. Other more complex issues concern the effect of hills, valleys and soil basins on the frequency content of ground motions and are still under research.

There are two main trends in the attempt to represent such defining characteristics in the seismic loading. The first is through the selection and/or scaling of natural ground motion records to match a target scenario and the second is through the direct generation of artificial time-histories. The pros and cons of the two options are discussed in the following sections.

2.2.1. Natural records

The use of natural (or "recorded") ground motions is the most common and preferable option for nonlinear response history analysis. Ground motion databases were scarce in the past, but in the recent years the number of recorded accelerograms has increased considerably owing to the (increasingly) large number of events that took place in well-instrumented countries. The limitation of using natural records is that they are consistent with a hazard scenario of a past event at a given site, and thus it is often difficult to find records consistent with the problem at hand, especially when considering the collapse of well-designed structures. The reason is that instrument recordings are relatively recent compared to the time-scale of earthquake occurrences. Therefore the ground motion databases contain primarily small-to-moderate records. Records of earthquakes with larger magnitudes at close distances are scarce thus posing an additional difficulty when a full-range assessment is sought. The common practice for circumventing this problem is to "scale" their amplitude in order to match, in terms of intensity, the corresponding hazard scenario that is often represented by a target acceleration spectrum over a range of periods. Scaling is a controversial issue that may introduce bias through systematically under- or over- estimating the actual response [12]. Usually the bias is small provided that the scaling is performed using an appropriate, or "sufficient", Intensity Measure (IM) [6].

Different approaches for selecting records using scaling without biasing the response have been proposed. Katsanos *et al.* [13] present a review of alternative selection procedures based on established methods for incorporating strong ground motion records within the framework of seismic design of structures. Watson-Lamprey and Abrahamson [14] show that scaling itself does not lead to biased results when the records are appropriately selected. Baker and Cornell [15] proposed selecting seismic records using the epsilon ‘ε’-method in order to reduce the bias. The epsilon “ε” of a ground motion record is defined as the number of standard deviations that separate its (pseudo) spectral acceleration value at a given period from the corresponding median estimate obtained from a GMPE, i.e. from a regression over multiple records. Other approaches for using scaled records in nonlinear dynamic analysis are presented by Aschheim *et al.* [16] and Kottke and Rathje [17].

Over the years, various methods that process ground motion databases in order to optimally select records and compile them in bins have been presented, e.g. [18,19]. Such algorithms may seek records that either individually, or on average, match a target spectrum (e.g. REXEL [20]). More elaborate procedures have been also presented. For example, Naeim *et al.* [21] proposed an approach based on an optimization algorithm in order to select a set of ground motions that minimizes the difference of the mean spectrum of the selected ground motions from the target design spectrum. Also, Jayram *et al.* [22], proposed a procedure that probabilistically generates multiple response spectra from a target distribution and then selects recorded ground motions whose spectra match the target spectrum. Recently, Katsanos and Sextos [23] proposed an algorithm for selecting ground motion records accounting for the variability of critical response quantities while also considering the properties of the structure studied.

Another process for using natural grounds motions to obtain records consistent with a given scenario is “spectrum matching”, i.e., the modification through signal processing of the natural records to reproduce a particular (typically the design) acceleration spectrum. There are numerous such methods and the quality of the results always depends on the specifics of the modification approach. For example, Abrahamson [24] and Hancock and Bommer [25] have proposed a wavelet-based algorithm to adjust recorded ground motions to match a specific target response spectrum. This algorithm is implemented in SeismoMatch [26] software.

2.3. **Synthetic records**

Some of the models and methods currently used for the simulation of seismic actions are discussed in [27]. Apart from natural ground motions, ground motion records can be also defined in the form of: (i) random processes, (ii) simulated accelerograms compatible with a design response spectrum, and (iii) synthetic accelerograms on the basis of a model of the earthquake source. This is an area of intensive research where many new methods and approaches are constantly emerging. Therefore, in this short survey, we explain some common methods used for simulating broadband and narrowband ground motions.

Random processes is a helpful tool for understanding the features of the maximum response of structures in the elastic range, while simulated records can be used to ensure consistency with the code requirements, since they are generated from a smooth design code-based response spectrum such as those obtained with the SIMQKE software [28,29]. The major shortcoming of these two methods is simply that they do not produce real seismic records and therefore cannot be adopted for the performance-based assessment of a given structure and a given site subjected to large inelastic deformations, since, contrary to linear elastic analysis, the number of cycles and their amplitude is important in this case. Regarding artificial accelerograms, the problems encountered from their use are discussed in [30]. Apart from SIMQKE, SeismoArtif [31] can be used to obtain artificial records.

Synthetic accelerograms can be obtained using various approaches. State-of-the-art derivations based on numerical models of the fault rapture and wave propagation from the source to the site have been developed. This approach is complex and includes intensive calculations, and therefore its application for engineering purposes is not recommended. However, there are regions (e.g. Los Angeles basin) for which physically sound synthetic records have been produced, e.g. [32]. Kinematic fault models are a more widely used option. Such models are based on the Green's function techniques, which follows the idea that the total motion is equal to the sum of the motions produced by a series of individual ruptures of many small patches on the causative fault [8]. Thus the fault is divided to a finite number of patches, while their sequential rupture is described by Green's functions. Such functions describe the time variation of the slip displacement of every patch. Typically all above processes have to be supplemented with an appropriate model of the soil effect so that the natural record is consistent with the local site soil conditions.

Another method for generating synthetic ground motions is based on the time-domain generation of transient stochastic processes. The idea is multiplying a stationary, filtered white noise signal with a function the describes the envelope of a ground motion. This multiplication transforms the stationary white noise to a non-stationary process. This concept has been adopted by Shinozuka and Deodatis [33] and also lies in the core of ARMA models (AutoRegressive Moving Average models), e.g. [34].

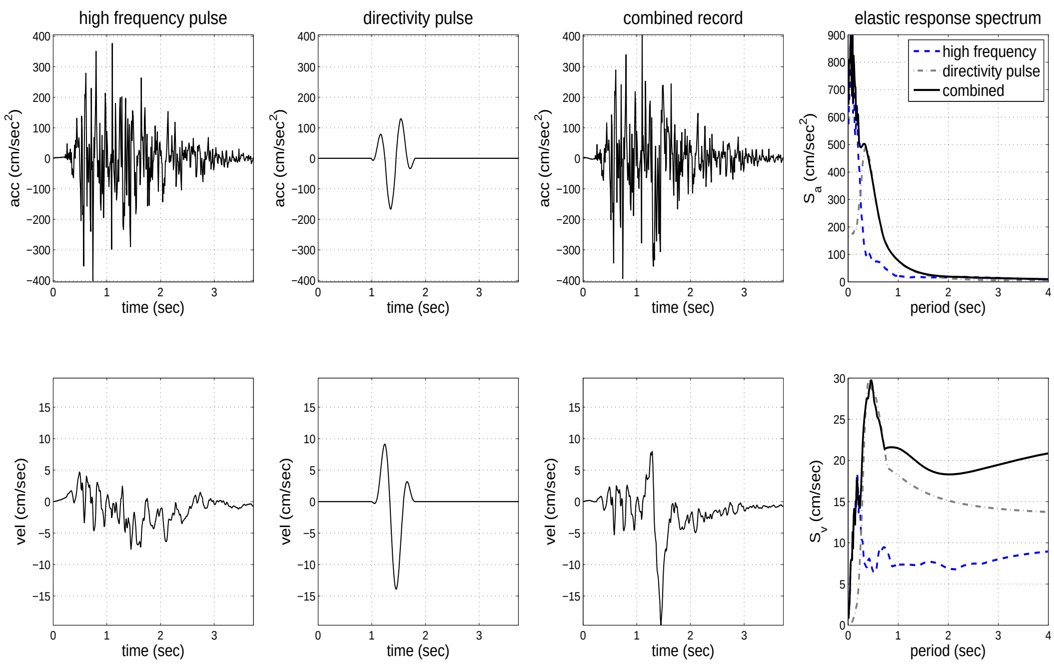


Figure 2. Generation of synthetic ground motion records. Upper row shows the acceleration and the bottom row the velocity time histories. The corresponding response spectra are shown at the further right column.

A rational and easy to implement procedure for producing synthetic records is the stochastic method [35]. In this case, the generation is performed in the frequency-domain (as opposed to the time-domain discussed above), using the ground motion radiation spectrum $Y(M_{w,}R,f)$, which is the product of quantities that consider the effect of source, path, site and instrument (or type) of motion. One of the products of $Y(M_{w,}R,f)$ contains the earthquake source spectrum, modelled with the ω-square model [36] or the specific barrier model [37,38]. The former is commonly used, but it is a point source model and hence not appropriate for near-fault problems, while it may also not be appropriate for large sources. Both problems are sufficiently handled by the specific barrier model.

By separating the radiation spectrum $Y(M_{w,}R,f)$ to its contributing components, the models based on the stochastic method can be easily modified to account for different problem characteristics. The stochastic approach consists of first generating a white noise (Gaussian or uniform) for duration predicted by an appropriate ground motion prediction equation (GMPE). The noise is then windowed and transformed into the frequency domain using an envelope function $w(M_{w,}R,t)$ and subsequently transformed back into the time domain. The application of the stochastic method can be carried out with the aid of the SMSIM program [35,39,40] that is freely available from the web. An extension of SMSIM is EXSIM [41]. EXSIM is able to consider information about the fault geometry and is appropriate for simulations of large earthquakes considering the sum of motions from subfaults distributed over a fault surface. The motions from each subfault are often given by SMSIM which is seen as a point-source simulation method. Boore in [42] compares the two programs and suggest simple modifications to SMSIM that render the two programs consistent.

When near-fault ground motions are required, the procedure suggested by Mavroeides and Papageorgiou [43] can be adopted in order to combine low frequency pulse models [43,44,45] with high-frequency synthetic ground motion records. The procedure for combining low and high frequency components consists of first obtaining the Fourier transform of both the high- and the low-frequency components. Subsequently the Fourier amplitude of the pulse is subtracted from that of the high-frequency component of the ground motion and a synthetic acceleration time-history is constructed so that its Fourier amplitude is that of the previous step and its phase angle is that of the high-frequency record. The final synthetic record is obtained by adding the pulse time-history. The outcome of this procedure is shown schematically in Figure 2, where the last column shows the corresponding acceleration and velocity response spectra. The velocity spectrum (bottom right figure) shows the impact of the directivity pulse, while looking at the third column, the effect of the pulse is clearly visible in the combined velocity time-history but difficult to discern when looking at the acceleration time-history.

Iervolino *et al.* [46] compared different procedures for obtaining sets of spectrum-matching accelerograms for nonlinear dynamic analysis of structures in terms of inelastic seismic response. The results of the analysis show that artificial, or adjusted, accelerograms may underestimate the displacement response when compared to original real records. The more recent work of Galasso *et al.* [47] also compared response estimations obtained with natural and synthetic records and suggest that, apart from some exceptions (e.g. short periods), synthetic ground motions are able to sufficiently match recorded ground motions.

The intention of the discussion above is to outline some major approaches for generating ground motions records and is by no means exhaustive. Other approaches or variations/improvements of the above can be found in the literature. Moreover, various software are available for generating ground motions, each following a different approach. For example, some of the methods referenced above are available in the openseource Broadband Platform software (BBP) [48], and also in SeismoArtif [31].

3. Structural modeling

Traditionally seismic design is performed assuming linear elastic analysis. The use of nonlinear analysis methods is more frequent for the assessment of existing structures, while their application is discussed in detail in recent guidelines e.g. ASCE/SEI-41[49]. Such analysis methods provide a highly efficient framework for design and assessment, since they allow more direct (high-level) design criteria and less simplifying assumptions. Moreover, despite the maturity of the finite element (FE) method, the seismic assessment of buildings is performed primarily with linear finite elements (e.g. beams or rods), while two- and three-dimensional finite elements are only rarely utilized, e.g. [50]. Apart from modeling the frame members with beam-column elements, the numerical model of a structure should also be able to adequately capture the response of other structural components that may considerably affect the overall capacity. Such components are, typically, the infill walls, the shear walls and other non-structural components. The discussion on these issues is beyond the scope of the paper. The reader may seek advice in the NIST GCR 10-917-9 [51] document. The effect of soil-foundation-structure interaction may also be significant. The effect of soil models of different complexity on the structural response is discussed by Assimaki *et al*. [52].

There are two major sources of nonlinear behavior: geometric and material nonlinearity. Geometric nonlinearity is essentially the effect of enforcing equilibrium at the deformed state, rather than the undeformed. Consider a single cantilever column subject to an axial compressive load P and a top horizontal displacement Δ, an additional moment of magnitude $P\times\Delta$ will be generated at the base of the column due to the horizontal displacement Δ. This effect is also known as "P-Δ" effect and it is becomes important for tall buildings that undergo excessive deformations. If instead of a top displacement Δ, we apply an initial bow-like imperfection δ at the member midpoint, then the compressive load P induces an additional moment at the midspan equal to $P\times\delta$, known as P-δ effect. $P\times\Delta$ effect affects globally the stability of the structure, while the P-δ effect is local since it depends on the imperfection (or the deformed state) of every member.

Structures collapse because the lateral seismic forces deteriorate the capacity of the structure and eventually they reach a state that they cannot carry their own gravity loads. Therefore, in earthquake engineering, the global P-Δ effects should be included in seismic assessment, especially for tall or flexible structures and/or for buildings near collapse. Local P-δ effects are typically taken into account only for members that are sensitive to imperfections and/or prone to buckling, most prominently braces [53]. Both effects are forms of geometric nonlinearity effect since additional stress resultants are generated as the structure deforms. Further discussion on the effect of geometric nonlinearities can be found in references [54,55,56,57].

From a computational standpoint, geometric nonlinearity can be accommodated either by an approximate 1st order assumption, where a linear geometric stiffness term is added to the element stiffness matrix, or via the accurate corotational approach [58]. The corotational approach offers significant advantages in terms of implementation in a FE analysis software [59].

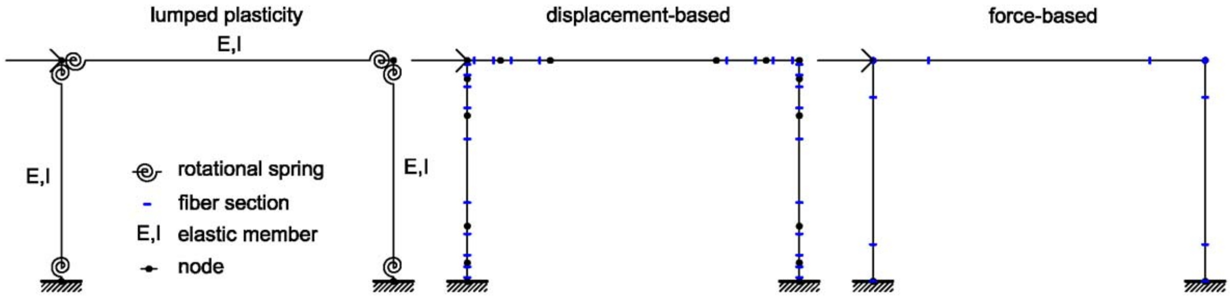


Figure 3 Finite element mesh requirements for the inelastic analysis of a portal frame. The lumped plasticity formulation (left) requires three elements and four rotational springs. The displacement-based formulation (middle) requires 15 beam elements and 30 Gauss integration sections (in total), and the force-based formulation (right) required three beam-column elements and six Gauss-Lobatto integration sections [60].

Material nonlinearity is considered of primary importance for the response of low/mid-rise structures. Significant advances have been realized particularly in FE simulation for the nonlinear material response of beam-column members that mainly falls in two categories: concentrated and distributed plasticity. In concentrated plasticity, the plastic deformations are “lumped” at the ends of a linear-elastic element and are based on the moment-rotation relationships of the end sections for a given axial force. On the other hand, distributed plasticity beam-column elements allow plastic hinges to form at any location along the member, while inelasticity is monitored in terms of stresses and strains, thus accounting for the axial-moment interaction. A schematic comparison is shown in Figure 3. Several commercial software packages are available and usually each follows a different formulation that the user/engineer must be aware in order to obtain reliable demand estimates.

3.1. **Section and member level**

As shown in Figure 3, estimating the inelastic response requires integrating the stresses calculated at appropriately selected cross-sections along the member. Moreover, in earthquake engineering often many studies are limited to studying the response of a single member and thus focus on the cross-section level and on developing its force-deformation or moment-rotation relationship. Therefore, it is essential to be able to sufficiently capture the moment-rotation relationship of a cross-section of given geometry and reinforcement. Numerically, moment-rotation relationships are obtained using the fiber approach, i.e. discretizing the cross-section to fibers that each follows its own material law. Quicker alternatives are also available [61], but usually they are not generally applicable and suitable for monotonic problems. Thus, fiber elements differ from lumped plasticity elements in the sense that they perform the integration over the fibers internally and therefore receive as input the parameters that define the stress-strain relationships of their fibers. On the other hand lumped plasticity elements receive as input the moment-rotation relationships of the cross-sections of interest, assuming that it is already known.

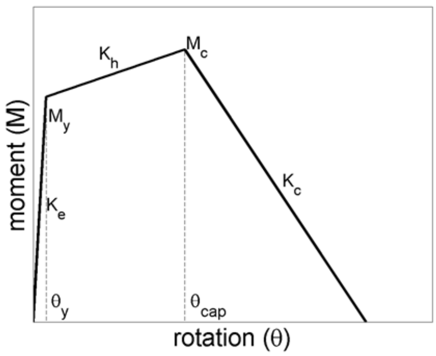


Figure 4 Typical moment-rotation curve according to FEMA P695 [62].

The section moment-rotation relationship can be defined assuming a backbone whose parameters are determined combing first principles and default values, or numerically performing section analysis (fiber analysis) with appropriate software. A typical backbone curve is shown in Figure 4. This backbone is discussed in the FEMA P-695 [62] guidelines. The curve essentially consists of three branches, an initial elastic (K_e), a hardening branch (K_h) and a descending branch with slope K_c, used to introduce degradation. Degradation, in general, can be either “in-cycle”, i.e. defined with a negative stiffness branch or “cyclic”. Cyclic degradation, usually refers to strength degradation (but not exclusively) at given displacement cycles due to capacity reduction during cyclic loading. The sensitivity of a building’s response to the parameters that define the backbone is demonstrated in [63], while a comprehensive discussion on cyclic and in-cycle degradation is also offered in [51].

Further developments refer to cyclic laws and cyclic degradation. Models that introduce cyclic stiffness degradation and pinching have been proposed. Such models either modify the path of the reloading branch [64,65] or introduce “pinching” [66-69]. Apart from linear or piecewise linear models, smooth hysteretic models have also been developed [70,71] in order to provide a continuous change of stiffness for the nonlinear springs. Huang and Foutch [72] performed a comparative study investigating the effect of different cyclic laws.

3.2. **Concentrated plasticity elements**

Concentrated plasticity is the approach commonly suggested in most design codes and guidelines [49]. The first concentrated plasticity beam elements were the parallel model of Clough and Johnston [65] and the series model of Giberson [73]. Preceding developments were based on the series model aiming at including the interaction of axial force and bending moment at the element ends (Figure 3) [74,75]. An efficient beam-column element is also the force-based lumped plasticity element proposed by Scott and Fenves [76]. This approach combines the benefits of concentrated and distributed plasticity elements since it uses a minimum number of integration sections maintaining all the advantages of force-based distributed plasticity elements which are discussed in the next subsection.

Lumped plasticity beam-column elements reduce the computational cost and the memory requirements and provide better numerical stability. Since the constitutive laws are expressed in terms of moment-rotations, more complex phenomenological relationships can be adopted compared to distributed plasticity elements where stress-strain relationships are used instead. This allows simulating complicated responses provided that the springs are appropriately calibrated, which is not always a trivial task. However, the concentrated plasticity approach relies on simplifying assumptions, restricting the region where inelastic deformations take place and/or separating the axial-moment interaction from the element behavior and thus the springs are calibrated assuming constant axial force during dynamic analysis.

3.3. **Distributed plasticity elements**

Distributed plasticity beam-column elements offer a more accurate description of the inelastic behavior since they allow inelastic deformations to be developed anywhere within the member (Figure 3, middle and right). These elements are also known as "fiber" elements, since usually the sections are divided to horizontal and vertical layers forming small areas, known as "fibers", where the strain and the stiffness parameters are evaluated. Fiber elements based on the classical FE theory use cubic Hermitian shape functions [77,78]. These elements are known in the literature as "displacement-based" or "stiffness-based" elements and their shortcoming is that they require a fine mesh of beam-column elements at the sections where inelastic deformations are expected to be high. This is necessary because the cubic shape functions assume a linear distribution of the curvature along the element which is not correct when the element end sections have yielded.

To overcome the problem stemming from the unknown distribution of curvature along the element, Mahasuverachai and Powell [79] proposed the use of flexibility-dependent shape functions. In this case, it is the distribution of forces (moments) that maters, while the element stiffness matrix can be easily calculated by summing the section flexibility matrices and inverting the resulting element flexibility matrix. This concept was extended by Kaba and Mahin [80] and later was improved by Zeris and Mahin [81] and Ciampi and Carlesimo [82]. These elements assume that the element is divided to equally spaced sections that allow the element to produce accurate predictions in the case of softening behavior, or in other words when the post-yield deformation path enters a segment with negative slope (softening). This formulation is also known as "force-based" or "flexibility-based". Following the previous concept, Spacone *et al.* [83] introduced a general mixed-type, fiber-based, beam-column element. At the integration sections (spaced according to the numerical integration method), this element always maintains equilibrium of both forces and deformations and converges to a state that satisfies the constitutive laws within a specified tolerance.

Compared to lumped plasticity, a major shortcoming of distributed plasticity elements is that they require more computing resources. Moreover, these elements are sensitive to the material law to be adopted and therefore it is more difficult to be calibrated with experimental results. Numerical instabilities may be encountered if criteria that introduce abrupt loss of capacity are adopted, e.g. when trying to predict collapse. On the other hand, distributed plasticity elements do not require any special calibration and can be easily adopted for sections that consist of different materials since they use stress-strain relationships suitable for each fiber material. A discussion on distributed plasticity elements can be found in [84], while comparisons between the force and the displacement-based are presented in [85,86,87]. All studies converge that this formulation is accurate and robust and therefore suitable for a wide range of applications.

A major shortcoming of fiber elements is that they are purely flexural. This means they do not account for the axial -shear-flexure interaction within the element because they are bounded by the kinematics of beam-column elements which allow the calculation of only one strain, the one parallel to the axis of the element. Thus, such elements are not suitable for the simulation of members subjected to high shear, e.g. walls, shear links, etc. On the other hand, lumped plasticity elements are able to overcome this problem by appropriately modifying the section moment-rotation relationship. As lumped plasticity elements are phenomenological, such calibrations sometimes are crude and not successful, especially for transient problems. Fiber elements that account the shear-axial-flexure interaction have been proposed both for steel [88,89] and reinforced concrete [90,91] structures.

Details and a further discussion on fiber beam-column and lumped plasticity elements and the limitations of each can be found in reference [60]. Moreover, Krawinkler *et al.* [92] also offer an interesting discussion, where apart from the issues discussed above, they pointed out that, "… there are great challenges in the use of fiber models including modeling of confinement, bond slip, rebar buckling and rebar fracture in reinforced concrete, and modeling of crack propagation, fracture, and local and lateral torsional buckling in steel. Since these phenomena are very difficult and often impossible to capture with fiber models, the need exists to place judgmental limits on strains (or curvatures) to account for the fact that these phenomena will govern the deterioration process when a structure approaches collapse.". The bottom line of the discussion is that whenever it is difficult of calibrate a phenomenological model, e.g. due to the lack of experimental results, fiber models are preferable and more fail-safe. In any case, whatever the final modeling choice (fiber or lumped), caution and engineering judgment are always essential in order to obtain meaningful response estimates.

Table 1: Analysis procedures for seismic design and assessment.

Category	Structural Model	Seismic Actions	Analysis Method
Linear-static	linear	equivalent load pattern	linear-static, force-control
Linear-dynamic (A)	linear	ground motion spectrum	modal response spectrum
Linear-dynamic (B)	linear	ground motion record	linear time-history
Nonlinear static	nonlinear	equivalent load pattern	nonlinear-static, displacement-control
Nonlinear dynamic	nonlinear	ground motion record	nonlinear time-history

4. Structural performance assessment

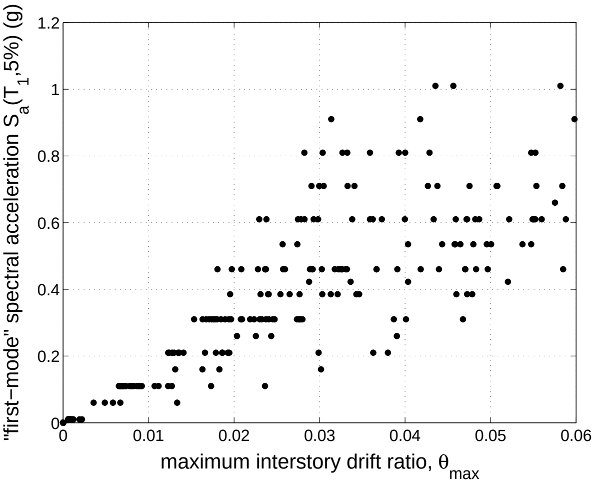
4.1. Analysis methods

Having an appropriate model for the structure and its earthquake loads, structural analysis is the next step. The choice of linear elastic or nonlinear modeling of the element force-deformation behavior and the inclusion, or not, of geometric nonlinearities, essentially dictate the selection of the appropriate analysis method. Modeling of structural mass distribution and the transient nature of earthquake loads allows the use of dynamic rather than static methods of analysis. Broadly speaking, four categories of analysis methods are available, each with its own modeling and solution algorithm requirements (Table 1) (ASCE 41-06 [93]): (a) nonlinear dynamic, (b) nonlinear static, (c) elastic dynamic (d) elastic static. In the following pages we are going to discuss them in detail, starting from the nonlinear methods that currently form the cutting edge of earthquake engineering research and working our way down to the simplest of choices. It is necessary to stress, that while each method is separately presented on its own, in engineering practice they are complementary rather than competing. Often in analysis, in order to gain confidence in the adequacy of a given numerical model, before proceeding to nonlinear dynamic analysis one has to start with an eigenvalue analysis and then move to a (nonlinear) static analysis. Experience has shown that if nonlinear dynamic analysis is performed without following those steps, modeling errors are likely to occur.

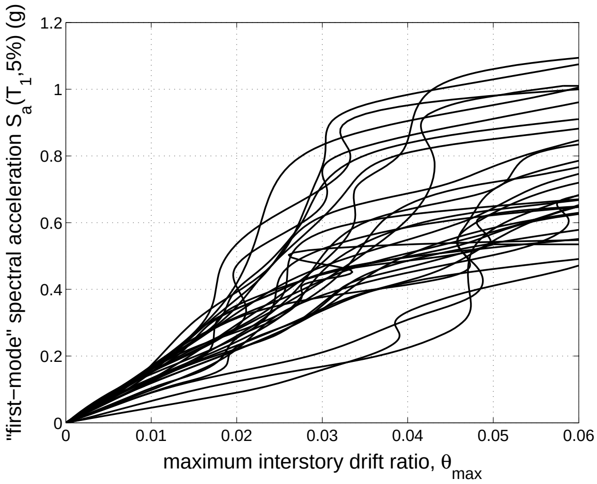
4.1.1. Nonlinear dynamic analysis

Nonlinear dynamic analysis is undoubtedly the most realistic and accurate analysis method available. The method is also referred as "nonlinear time-history analysis", "nonlinear response history analysis", or according to ASCE 41-06 [49] as "nonlinear dynamic procedure" (NDP). Earthquake loading is applied as a natural or a synthetic ground motion on a structural model that incorporates elements with inelastic (inelastic and nonlinear are typically used interchangeably for seismic applications) force-deformation relationships and at least a first-order approximation of geometric nonlinearities, i.e. P-Delta effects. The propagation of the ground motion throughout the structure generates complete response histories for any quantity of interest (e.g. displacements, stress resultants) leading to a wealth of data. While different levels of complexity are possible due to modeling choices (see earlier sections), different ground motion records (stemming from different seismic events, or even different recording stations) will produce demands that vary considerably. This record-to-record variation dominates the application of dynamic methods. Therefore, in order to get reliable response estimates an appropriately selected set of several ground motion records is necessary, since a single time-history analysis is of limited practical use. Furthermore, one or more levels of seismic intensity, typically corresponding to one or more levels of the intensity measure (section 2.1) may need to be employed to investigate structural behavior at different regimes of response or damage (e.g. elastic, post-yield or near-collapse). Thus, nonlinear dynamic analysis procedures may be categorized into the narrow- and broad-range assessment categories.

In most practical design/assessment situations, only a narrow, single-point estimate of structural response is required, represented for example by a single horizontal "stripe" of analysis results for a given value of the IM in Figure 5a. This is consistent with current seismic codes that only provide a design hazard spectrum (typically at an exceedance probability of 10% in 50yrs) and require checking that a structure shall not sustain significant or life-threatening damage at such a level of intensity. Thus, seismic codes (e.g. ASCE 7-10 [93], EN1998 [94]) prescribe using ground motion records that match (or exceed) the design spectrum in the period range of interest. If 3 – 6 records are employed, the overall maximum of the 3 – 6 recorded peak responses is taken as the structural demand. For 7 records and above, the mean of the peak responses can be employed. Widespread use of such rules at their bottomline of 3 or 7 records has been the source of criticism for potentially unconservative results. Bradley [95] has shown that explicitly including the statistical error in the response estimates can convey a much more satisfactory level of confidence in accordance with the number of records employed.



(a)



(b)

Figure 5 (a) Discrete analysis points, and (b) their interpolation into continuous IDA curves for a 9-storey steel moment-resisting frame. The EDP ($\theta_{\max}$) is the maximum drift over time and over all stories and the IM is $S_a(T_1)$.

Broad-range evaluation methods are the mainstay of current research due to their use in probabilistic assessment frameworks for performance evaluation. This will be apparent in later sections, where probabilistic methods will be discussed. The goal is to employ a sufficient number of ground motion records that are widely spread in terms of intensity to achieve a broad characterization of the relationship of the intensity measure (IM) versus the engineering demand parameter (EDP). Any structural response variable of interest may be used as the EDP, for example story drift, plastic hinge rotation, or column shear. Typical choices for the IM are quantities that can be related back to the seismic hazard analysis and an appropriate exceedance probability. A prime example is the 5% damped, first-mode pseudo-spectral acceleration $S_a(T_1)$. As elaborated in earlier sections, proper IM selection is an important aspect in the context of conditional probabilistic frameworks.

Ground motion synthesis or ground motion selection, modification and scaling can be employed to provide the seismic input for dynamic analysis. Different modes of employing such record suites come with different needs for treating and postprocessing the results. Incremental Dynamic Analysis (IDA, [96,97]) employs a record suite that is scaled to cover all needed levels of intensity, from elasticity up to global dynamic instability. The resulting discrete points (one per each dynamic analysis) in the IM-EDP plane (Figure 5a) are interpolated to form the distinct IDA curves (Figure 5b), showing an initial elastic behavior that terminates at a characteristic flatline when global collapse (equivalent to numerical non-convergence in a well-executed analysis) is encountered (Figure 6a). The single records IDAs are then summarized to their 16/50/84% IM given EDP or EDP given IM fractile curves (Figure 6b). These are used to characterize the probabilistic distribution (typically lognormal mean and standard deviation) of IM|EDP or EDP|IM. A similar method is stripe analysis, where records are selected and scaled to common levels of IM to create characteristic horizontal stripes of EDP given the IM level, similar to Figure 5a. The third option in current use is cloud analysis, where one or more suites of ground motions may be selected or scaled to correspond to different average levels of IM, giving a distinct impression of small clouds of points in the IM-EDP plane that can be typically fitted by regression to obtain a full probabilistic characterization [98].

Of the above three methods, the continuous representation of the IM-EDP relationship offered by IDA, rather than the stripes or clouds of the others, makes it an excellent vehicle for understanding the seismic behavior of structures, but also the capabilities of simpler analysis techniques. For example, to get a sense of perspective, the 3 – 7 record point-estimation method stipulated by modern seismic codes simply corresponds to a single stripe or a small cloud centered at the design spectral acceleration value in Figure 5. Such desirable properties have spawned considerable effort in simplifying the evaluation of IDA curves, for example through the progressive record lists [99,100] and by exploiting its connection with the static pushover curve that will be discussed in the following section.

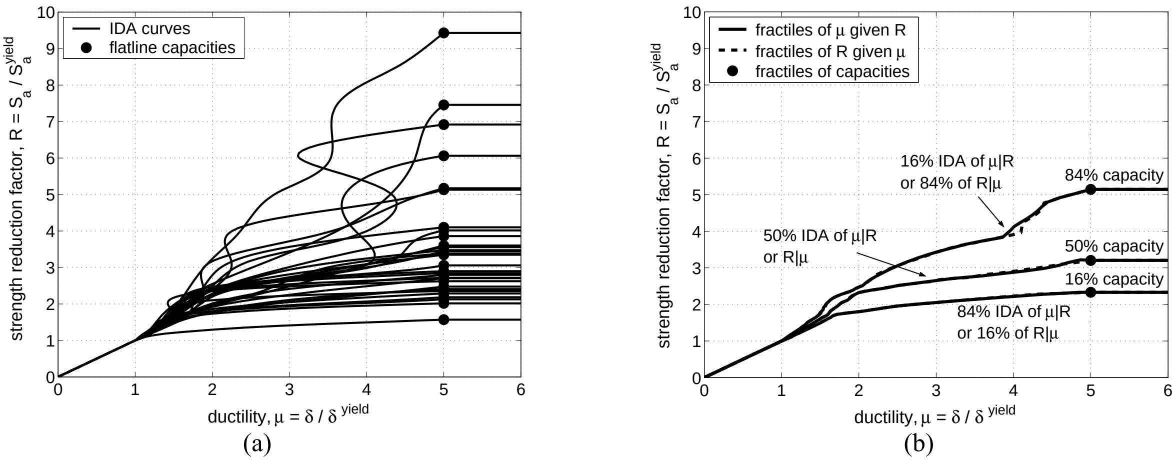


Figure 6 (a) Forty IDA curves, and (b) their summarization into 16/50/84% fractiles for an SDOF system. The EDP is ductility and the IM is $S_a(T_1)$ normalized by its yield-level value (a ratio equal to the elastic seismic force demand over the yield strength of the oscillator, known as the strength reduction factor R). The x% fractiles EDP|IM curves are practically identical to the (1-x)% IM|EDP curves [101].

4.1.2. Nonlinear static analysis

To mitigate the resource-intensive nature of nonlinear dynamic analysis of MDOF models, simpler solutions may be sought by employing instead equivalent nonlinear SDOF models and pre-recorded estimates of their displacement response. In essence, this is the premise of nonlinear static procedures (NSPs) that were probably first suggested by [102] and since then have seen wide-spread adoption by current seismic codes [49,94] in an effort to introduce them in engineering practice. According to the description of Fajfar (N2 method [103]) two nonlinear models are required to accomplish the suggested simplification from nonlinear dynamic to nonlinear static analysis. One is for the MDOF model that is analyzed in a static pushover by applying a lateral load pattern typically approximating the first eigenmode. In the classic setting, this model will give the base shear versus roof displacement "capacity curve", which is then employed to define the period and force-deformation relationship of the equivalent SDOF that will serve as second model. The capacity curve refers to a characteristic control node usually at the building's roof and the lateral load pattern is incremented, "pushing" monotonically the structure in the inelastic zone.

Having the second model (SDOF) in our disposal, the demand is obtained as the target displacement of the characteristic node. Instead of running dynamic analyses of the SDOF model to estimate its IM-EDP relationship, pre-computed estimates (or relationships) can be used. Such displacement estimates are typically offered by R-μ-T (or C_R-μ-T) regression relationships of the form [104]:

$$C_R(T,R) = \frac{\mu(T,R)}{R} \tag{1}$$

where R is the ratio of the (elastic) seismic force, divided by the system base shear strength at (nominal) yield, μ is the ductility, expressed as the control node displacement divided by its (nominal) yield value, T is the equivalent system period and C_R is the corresponding inelastic displacement ratio (ratio of inelastic over elastic displacement). R-μ-T relationships are essentially regression equations of the "central" (mean or median) IDA curve of the equivalent SDOF system, shown for example in Figure 6b. By having such relationships available, the so called "target" or "performance point", representing the central value of the control node displacement δ_t, may be located on the capacity curve, enabling a fast approximation of the expected structural response:

$$\delta_t = \Gamma C_R(T,R)\frac{T^2}{4\pi^2}S_a(T) \tag{2}$$

where Γ is the first mode participation factor, evaluated for the mode shape having been normalized by the control node displacement. Eq. (2) is essentially the basis of the displacement coefficient method of ASCE 41-06 [49] for static pushover analysis.

Conceptually, the static pushover is directly related to the nonlinear dynamic analysis through the SDOF simplification. Vamvatsikos and Cornell [96] have shown that the static pushover curve indeed represents an intrinsic property of the system that corresponds to its dynamic response in the IM-EDP plane (Figure 7): Different segments of the capacity curve correspond to different dynamic response segments (median IDA curve) with "hardening" or "softening" behavior vis-à-vis the elastic.

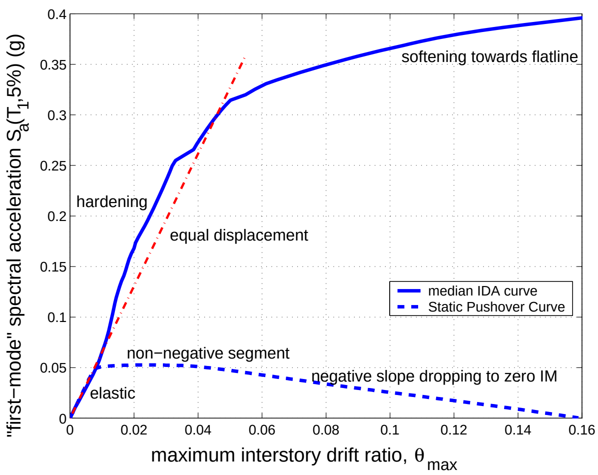
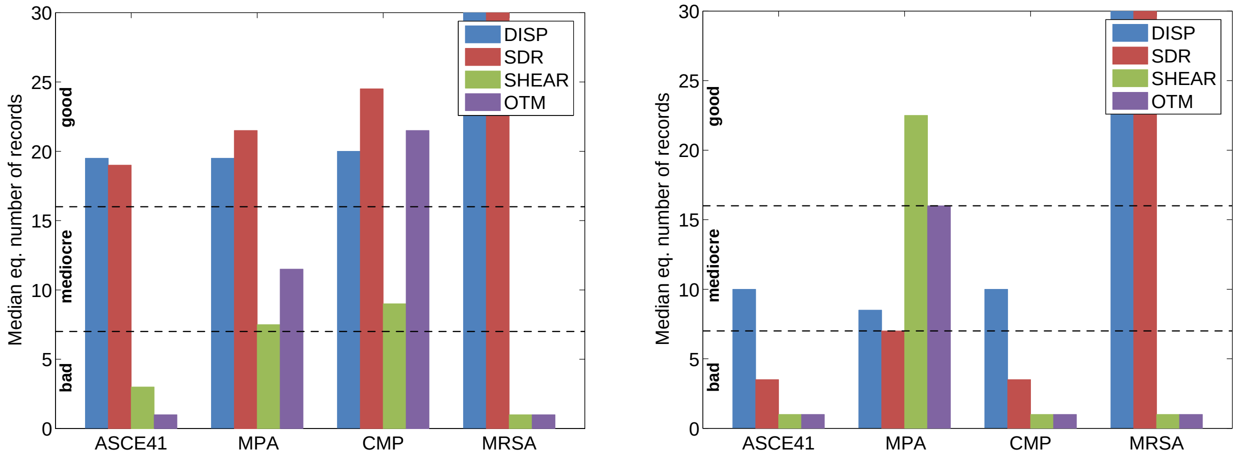


Figure 7 The median IDA versus the static pushover curve for a 20-story steel moment-resisting frame, showing the correspondence between different regions of response. The post-yield non-negative stiffness range translates to an "equal displacement" nearly-elastic response, while the loss of strength eventually leads to global collapse [96].

Pushover analysis comes with many advantages [105], most prominently its apparent simplicity: One curve that gracefully explains how the structure behaves under increasing loads, showing the sequence of element failures that lead to eventual collapse. Unfortunately, while to the untrained eye it is easy to seem that the single answer provided is the truth, its theoretical basis is not robust. NSP's simplicity is achieved mainly by (a) ignoring higher modes, (b) neglecting multiple potential collapse mechanisms, (c) neglecting ground motion record-to-record variability (apparent on the IDA curve of Figure 5), and (d) fitting standardized capacity curve shapes (typically elastic-perfectly-plastic) to a complex real shape. All in all, standard pushover methods are considered reliable for low/mid-rise buildings without irregularities. Significant research has, thus, been directed to partially mitigate such deficiencies and better relate the static pushover results with the actual dynamic response.

Several attempts to propose improved pushover methods have focused on including modes higher than the first, resulting to the so-called multi-mode pushover methods. Among them the most well-known method is the Modal Pushover Analysis (MPA, [106]) that subjects the structure to several additional nonlinear (or linear) static analyses, each under a load pattern proportional to a higher mode. The results (EDP values) from each separate analysis are combined through a square-root-sum-of-squares (SRSS) rule to provide the final force and/or deformation estimates. Subsequent publications have further refined the method, for example by improving element force estimates [107]. A different approach is offered by the Consecutive Modal Pushover (CSP, [108]) that uses only a single analysis where the load pattern is adjusted in the course of incrementing pushover loading to sequentially include in the lateral load pattern modes higher than the first. More recently, an extension of the N2 method has been presented including higher mode effects either/both in plan [109] and elevation [110]. This method uses the envelope of nonlinear first-mode pushover estimates and linear elastic results for higher modes.

Enforcing a constant lateral load pattern has often been criticized as a source of error in NSP, since this practice essentially biases the collapse mechanism. Attempts to resolve this problem have often revolved around the use of adaptive load patterns that utilize instantaneous eigenmode information at each displacement increment to modify the applied lateral load pattern [111,112]. Compared to MPA, these methods combine with SRSS the lateral load pattern, while MPA combines the heightwise profile of the demand (e.g. storey displacements, drifts). A twist to this idea is offered by the Displacement Adaptive Pushover (DAP, [113]), whereby an adaptive displacement pattern, rather than a constant load, pattern is used. Newer techniques attempt to combine the advantages of adaptive methods with multi-mode procedures [114]. Nevertheless, such improvements to the standard NSP tend to come at the cost of increased complexity, unless specialized software is available, while they do not always offer considerably improved results.



(a) 4-story RC building (b) 8-story RC building

Figure 8 The accuracy in three NSP predictions (ASCE-41, MPA, CMP) and modal response spectrum analysis (MRSA) expressed in terms of the equivalent number of records needed for nonlinear dynamic analysis to achieve the same accuracy. For each building and EDP type, i.e, displacements (DISP), story drifts (SDR), story shears (SHEAR) and story overturning moments (OTM), the median number of "required" records is calculated over all stories for two intensity levels [115]. A value lower than 7 is considered to be lower than code standards, while one higher than 16 offers good accuracy.

Improving the SDOF to MDOF connection is another important research field that has great impact in the accuracy of NSPs. In terms of estimating the target point displacement, two competing methodologies are used, namely, displacement modification and equivalent linearization. In the first case, R-μ-T relationships [116] are employed in conjunction with Eq. (2) to directly estimate the inelastic displacement of the target point. In the second case, an equivalent linear system is used to predict the target point displacement by lengthening the period and increasing the damping of the original hysteretic SDOF [117]. Through the work of Miranda and Ruiz-Garcia [118] and the FEMA-440 guidelines [119] it has been shown that both methods can produce similar results as long as no direct physical interpretation is stipulated. Soft soil [120,121] and near fault sites [122,123] need their own R-μ-T relationships, attracting their fair share of attention. With the majority of existing literature being on bilinear elastic-perfectly-plastic or elastic-hardening capacity curves, Vamvatsikos and Cornell [101] offered relationships for quadrilinear capacity curves with optional negative stiffness to estimate not just the mean but also the 16/50/84% inelastic response. This allowed for the first time the capability of introducing variability into NSPs and estimating global dynamic instability from static pushover results. Moreover, De Luca *et al.* [124] proposed accurate fitting rules whereby a curved realistic pushover shape can be near-optimally approximated by piece-wise linear shapes to allow the accurate estimation of the equivalent SDOF properties. Finally, there have been several attempts to exploit the apparent connection of the capacity curve and IDA to offer results that are more consistent with dynamic analysis. Of particular importance has been the prediction of global dynamic instability that has been shown to be possible within a reasonable error margin using a single-mode static pushover [125-128].

Despite the wealth of improvements brought in recent years, NSP remains a procedure that is fraught with simplified assumptions that introduce errors. Thus, significant criticism has been directed to its shortcomings that even the most advanced of its variations do not seem able to resolve satisfactorily [62,115,129]. In general, nonlinear static methods cannot reproduce the multiple inelastic mechanisms that may appear or the interaction and timing of modes that produce the peak response values in dynamic analysis. Thus, regardless of their sophistication there will always be a certain error threshold that they will not be able to transcend. Fragiadakis *et al.* [115] have quantified such errors vis-à-vis nonlinear dynamic analysis through the equivalent minimum number of records that the dynamic method would need to achieve the same degree of accuracy as the NSPs. Errors equivalent to less than 7 records can be considered subpar according to typical code requirements. Figure 8, summarizes these results for two regular reinforced-concrete moment resisting frames with four and eight stories, comparing the classic pushover (ASCE/SEI 41-06 version), MPA, CSP and the simple elastic modal response spectrum analysis (MRSA). The accuracy is expressed in terms of the equivalent number of records needed for nonlinear dynamic analysis to achieve the same accuracy. For each building and EDP type, i.e, displacements (DISP), story drifts (SDR), story shears (SHEAR) and story overturning moments (OTM), the median number of "required" records is calculated over all stories for two intensity levels [115]. A value lower than 7 is considered to be lower than code standards (denoted as 'bad'), while one higher than 16 offers good accuracy (denoted as 'good'). In general, force quantities are more difficult to capture than deformation ones, while as the height increases larger errors invariably appear. As a disclaimer, note that the completely unsophisticated MRSA in Figure 8 seems to provide the perfect solution for displacement-based quantities, yet this is only due to the absence of any damage localization in the newly designed buildings studied in [115].

4.1.3. Linear methods

Although simple elastic linear methods may seem antiquated by modern research standards, they define, by overwhelming majority, the current standard-of-practice for seismic design. Despite many efforts to offer a design methodology on a nonlinear static or dynamic basis [130], the vast majority of practical earthquake engineering work is still done at the elastic level using either: (a) equivalent linear static analysis, whereby a fixed lateral load pattern is applied to an elastic structural model (paralleling the application of NSPs) and (b) modal response spectrum analysis, where the modal responses are combined (typically via SRSS) to estimate the peak MDOF response (e.g., [120]). In the latter case, either a design spectrum or actual ground motion records may be used to define the seismic loading. Inelastic response is universally considered by dividing the elastic seismic loads (i.e., design spectral acceleration values) with a code-specified reduction R (or behavior q) factor that is meant to represent the ductility and overstrength of a yielding system. Although such factors are typically a compromise between theory and practice, an accurate and theoretically consistent process for deriving values of R has been established based on IDA via the FEMA P-695 guidelines [62].

While little recent research has been directed in the way of elastic methods, current advancements in nonlinear analysis have helped to shed some light into the premise of using elastic results to capture nonlinear behavior. Specifically, IDA has shown how the so-called "equal displacement" rule (i.e., the observation that elastic and mean inelastic displacements of elastoplastic oscillators are often similar [116]) that is valid in the medium and long period range (approximately $T > 0.6$s), means that elastic methods perform relatively well in predicting displacements, as long as the structure is not in the negative stiffness phase (see Figure 7). This is essentially the reason behind the good performance shown by the elastic MRSA in Figure 8.

5. State-of-art assessment/design frameworks

In contrast to typical static (or quasi-static) loading situations, the infrequent nature of seismic loads and their nearly unbounded magnitude invariably introduces the dimension of time. Thus, the basic safety inequality assessment of action versus resistance does not provide an adequate description of seismic safety. Given that when a ground motion violates the inequality we cannot necessarily assume that the building has failed, the real question is how often is such an event going to happen in the lifetime of the structure, and what consequences will this violation of the safety inequality (or failure) have.

Furthermore, nowadays structural assessment is not only about estimating the structural response. Engineering quantities such as displacements, accelerations, plastic rotations, shear forces and moments make very little sense to stakeholders (e.g., building owners, insurance companies or governments). Non-engineers typically communicate in financial terms, such as the net present value of an investment. This shift in the focus of assessment marks the advent of modern "performance-based" (or "consequence-based") earthquake engineering that has essentially become the mainstay of contemporary earthquake research. In this section, we discuss important elements of such methodologies, focusing on the measurement/definition of structural performance over the lifetime of the structure.

5.1. **Deterministic vs Probabilistic frameworks**

The variable nature of earthquake loading and the limited knowledge about the complex properties of soil and structure means that probabilistic concepts have become essential for earthquake assessment. For reasons of book-keeping mainly [131], variability is categorized in two broad groups in relation to its reducibility: *aleatory* randomness characterizes the natural variability, e.g., of ground motion, while *epistemic* uncertainty is related to our incomplete knowledge, for example, of the material properties of an existing structure. While aleatory sources cannot be removed, epistemic sources can have their variability reduced by undertaking appropriate tests, although this is often economically or practically infeasible.

Typically, seismic intensity for a given mean annual frequency, structural demand for a given intensity and structural capacity/resistance to inelastic deformation are modeled by lognormal random variables, characterized by heavy right tails and large probabilities of exceeding values to the right of the mean. Such distributions are represented by the mean and standard deviation of their logarithmic values, or equivalently by their median μ and dispersion β, the latter being numerically very similar to the coefficient of variation (for values less than 0.7). Natural record-to-record dispersion is typically in the order of 30-40% at least, compounded with seismic hazard values whose uncertainty exceeds 100%. Thus, accurately quantifying and propagating such sources of variability all the way to structural response and performance estimates has become an important issue. While the consideration of multiple ground motion records, e.g., through IDA, may take care of the record-to-record variability, structural model uncertainty is still an open problem in earthquake engineering [63,131-135].

Nevertheless, seismic assessment is at its core a discipline that is practiced by professional engineers and it has deep roots in the tradition of infrastructure design over many decades. Therefore, seismic codes and guideline documents typically emphasize a deterministic approach where probabilistic aspects are roughly (and hopefully conservatively) approximated through "appropriate" choices of load levels and safety factors. Thus, it is no wonder that all codified nonlinear static procedure (NSP) approaches essentially lack any trace of variability. The obvious shortcomings and constraints placed by such simplifying assumptions have been recognized over the years, contributing to the emergence of performance-based earthquake engineering, where, among others, proper characterization of (the distribution of) structural response, damage and loss are essential features. Perhaps the best introduction to this never-ending discussion is offered by Bazzurro et al. [7] who compare the three fundamental frameworks for assessing structural performance, comparing the deterministic NSP against the conditional and the non-conditional probabilistic approaches (see also [130]).

Conditioning on the value of the intensity measure (IM), as already discussed, effectively separates the tasks of the seismologist and the structural engineer. At the cost of selecting a sufficient IM that can incorporate all the necessary seismological information without biasing the analysis, this also has the effect of massively reducing the number of required structural analyses. It is no wonder, then, that conditional approaches have dominated the scene from the very start. Arguably, the two most prominent such frameworks are offered by the PEER Center and the SAC/FEMA guidelines. Together with the Monte Carlo approach, these will be the focus of the following three sections.

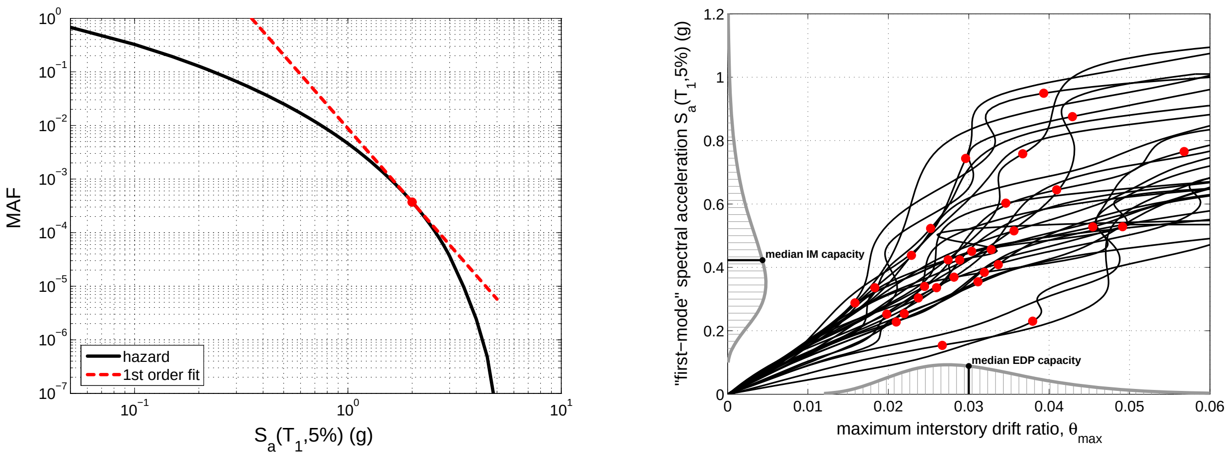
5.2. The PEER framework

Adopting a Poisson model for earthquake events allows expressing the structural performance via annualized earthquake-related losses. These may be quantified, e.g., by the triptych of repair costs, downtime and casualties that has been adopted by the Pacific Earthquake Engineering Research (PEER) Center in the form of the Cornell-Krawinkler framing equation [137]:

$$\lambda(DV) = \iiint G(DV \mid DM) \cdot \left|\mathrm{d}G(DM \mid EDP)\right| \cdot \left|\mathrm{d}G(EDP \mid IM)\right| \cdot \left|\mathrm{d}\lambda(IM)\right| . \tag{3}$$

DV is a single or a vector of decision variables, such as cost, time-to-repair or human casualties that are meant to enable decision making by stakeholders. DM represents the damage measures, typically discretized in a number of Damage States (e.g. red/yellow/green) of structural or non-structural elements and building contents. EDP contains the engineering demand parameters such as interstory drift or peak floor acceleration and IM is the seismic intensity, for example the 5%-damped first-mode pseudo spectral acceleration $S_a(T_1,5\%)$. The function $\lambda(y)$ provides the mean annual frequency (MAF) of exceedance of y, while $G(x)$ is the complementary cumulative distribution function (CCDF) of variable x.

The simplified formulation of Eq. (3) has received some criticism [138], yet its usefulness has been proven in many ways in the past years. One of its most important applications is the probabilistic estimation of losses from seismic events [139]. This has originally appeared in the form of the assembly-based vulnerability method of Porter *et al.* [140] for assessing repair losses and downtime. It was further improved and integrated with the PEER methodology by Aslani and Miranda [141] who also incorporated the dichotomy of collapse versus non-collapse. Finally, Ramirez and Miranda [142] provided the third generation loss assessment framework by adding the influence of residual displacements on the probability of demolition and the associated decommissioning costs. This is a rapidly evolving area of research and many improvements are expected to appear over the next few years.



(a) (b)

Figure 9 (a) S_a seismic hazard curve of Van Nuys, CA for T = 2.35s and its power law fit, (b) IDA curves, collapse points and EDP_c, IM_c lognormal distributions for a 9-story steel frame (from [143]).

5.3. The SAC/FEMA framework

Despite the usefulness of the comprehensive PEER approach, defining performance without involving any *DV* or the closely related *DM often* makes more sense for practice. Engineering quantities may be preferable, especially when working at the level of a design office, to discern the superior structure. This may be best achieved by moving to the familiar territory of limit-states: Let *DV* and *DM* be indicator variables that become unity when a given limit-state (LS) is exceeded, Eq. (3) simplifies to estimate λ_{LS}, the MAF of violating the limit-state [98]:

$$\lambda_{\mathrm{LS}} = \int_0^{+\infty} \left[\int_0^{+\infty} F(EDP_c \mid EDP) f(EDP \mid IM) \, \mathrm{d}EDP \right] \cdot \left| \frac{\mathrm{d}\lambda(IM)}{\mathrm{d}IM} \right| \mathrm{d}IM \tag{4}$$

where F is the cumulative distribution function (CDF), f the probability density function (PDF) and EDP_c is the limit-state capacity expressed in terms of the EDP. The nested integral is often represented as $F(IM_c|IM)$, the CDF of the IM capacity for the limit-state, better known as the fragility function. In general, EDP_c and IM_c are intimately related probabilistic quantities that characterize a limit-state for a given structural system, best visualized on the IM-EDP coordinates of the familiar IDA curves (Figure 9b).

Eq. (4) may be less complex than the PEER framework, yet it is not simple enough for practical application. The breakthrough came with the work of Cornell *et al.* [144] who, motivated by the failures observed in steel frames during the 1994 Northridge earthquake, developed a closed-form solution for the SAC/FEMA guidelines [145,146]. Therein, the hazard curve function $\lambda(IM)$ is approximated by a linear fit in log-log coordinates (see Figure 9a) with a slope of k. If the EDP demand is lognormal with a conditional median of:

$$EDP_{50} \cong a \cdot \left(IM\right)^b \tag{5}$$

and dispersion β_d, while the EDP capacity is also assumed lognormal with parameters EDP_{c50} and β_c, Eq. (4) becomes:

$$\lambda_{\mathrm{LS}} \cong \lambda\left[\left(\frac{EDP_{c50}}{a}\right)^{\frac{1}{b}}\right]\exp\left[\frac{k^2}{2b^2}\left(\beta_d^2+\beta_c^2\right)\right]. \tag{6}$$

The effect of epistemic uncertainty of demand and capacity can also be incorporated either by appropriately inflating the argument of the exponential to estimate either an overall mean, or value that will not be exceeded with a given confidence.

Such expressions offer a direct estimate of structural performance by capitalizing on the power of nonlinear static or dynamic analyses [98] and PSHA to offer useful intuition into the effect of hazard, structural behavior and associated uncertainties on the estimated MAF of limit-state exceedance. The SAC/FEMA formulas have thus become the state-of-art in the attempt to provide a performance basis for seismic design and assessment. Subsequent work, though, has shown them to be prone to errors [147], especially when the curvature of $\lambda(IM)$ is significant [3]. A biased fit that better matches the hazard to the left of the median capacity [148], or, even better, a second-order fit paired with improved closed-form expressions [149] can reduce such errors substantially, opening the road for wide-spread implementation.

5.4. **Monte Carlo methods**

The advancements in reliability theory during the last twenty years and the attainment of more accurate quantification of the uncertainties associated with system loads and resistances have stimulated the interest in the probabilistic treatment of systems [150]. The reliability of a system or its probability of failure is an important factor in the design procedure since it investigates the probability of the system to successfully accomplish its design requirements. Reliability analysis leads to safety measures that a design engineer has to take into account due to the aforementioned uncertainties. Although from a theoretical point of view the field has reached a stage where the developed methodologies are becoming wide-spread, from a computational point of view serious obstacles have been encountered in practical implementations. First and second order reliability methods [29,151,152] that have been developed to perform reliability analysis, although they lead to elegant formulations, they require prior knowledge of the means and variances of the component random variables and the definition of a differentiable limit-state function. On the other hand the Monte Carlo simulation (MCS) method is not restricted by the form and the knowledge of the limit-state function but is characterized by the high computational cost.

Monte Carlo simulation is applied in stochastic mechanics when an analytical expression of the limit-state function is not attainable. This is mainly the case in problems of complex nature with a large number of random variables, where all other methods are not applicable. In reliability analysis, the limit-state probability can be written as:

$$p_{ls} = \int_{g(\boldsymbol{x})\geq 0} f_x(\boldsymbol{x})d\boldsymbol{x} \tag{7}$$

where $f_x(\boldsymbol{x})$ denotes the joint probability function of the random variables, the limit-state function $g(\boldsymbol{x})<0$ defines the safe region and $\boldsymbol{x}$ is the vector of the m random variables. Considering that MCS is based on the theory of large numbers (N_∞) an unbiased estimator of the limit-state probability is given by:

$$p_{ls} = \frac{1}{N_\infty} \sum_{j=1}^{N_\infty} I(\boldsymbol{x}_j) \tag{8}$$

where $\boldsymbol{x}_j$ is the j^{th} vector of the random system parameters, $I(\boldsymbol{x}_j)$ is an indicator if the j^{th} simulation violates or not the limit-state function, and is defined as:

$$I(\boldsymbol{x}_j) = \begin{cases} 1 & \text{if} \quad g(\boldsymbol{x}_j) \geq 0 \\ 0 & \text{if} \quad g(\boldsymbol{x}_j) < 0 \end{cases} \tag{9}$$

In order to estimate p_{ls} an adequate number of N_{sim} independent random samples is generated using a specific, uniform probability density function of the vector $\boldsymbol{x}$. The value of the limit-state function is computed for each random sample $\boldsymbol{x}_j$ and the Monte Carlo estimation of p_{ls} is given in terms of sample mean by:

$$p_{ls} \cong \frac{\mathrm{N_H}}{\mathrm{N_{sim}}} \tag{10}$$

where N_H is the number of simulations where the limit-state function is violated while N_{sim} is the total number of simulations.

The basic MCS is simple to use and has the capability of handling practically every possible case regardless of its complexity. However, the computational effort involved becomes excessive due to the large sample size N_{sim} required, especially when seeking small failure probabilities. To reduce the computational effort more elaborate sampling methods, known as variance reduction techniques, have been developed, e.g. importance sampling [153] or subset simulation [154].

5.5. **Life-Cycle Cost Assessment**

In construction industry decision making for structural systems situated in seismically active regions, requires consideration of the damage cost and other losses resulting from earthquakes that will occur during the lifespan of a structure. Thus, life-cycle cost analysis (LCCA) can be an essential component of the design process used to control the initial and the future cost of building ownership. Contrary to methods based on annualized quantities, such as the MAF (e.g., the PEER framework presented earlier), which essentially present a time-independent or time-averaged view, LCCA can offer the missing resolution in time. At the cost of higher complexity, this is a non-trivial improvement that can become important for time- and age-sensitive problems.

In early 1960s LCCA was applied in the commercial area and in particular in the design of products considering the total cost of developing, producing, using and retiring. The introduction of LCCA in construction industry was made in the field of infrastructures as an investment assessment tool. In particular, in early 1980s it was used in USA as an appraisal tool for the total cost of ownership over the lifespan of an asset [155,156]. Later, in view of large losses due to extreme hazards, like earthquakes and hurricanes, there was a need for new design procedures of facilities that could lead to life protection and reduction of damage and economic impact of such hazards to an acceptable level [157]. In this context, LCCA was introduced in the field of constructions as a complex investment appraisal tool incorporating a structural performance criterion [158]. Decision-making for structural systems situated in seismically active regions, requires consideration of the damage and the cost of other losses resulting from earthquakes occurring during the lifespan of the structure. A considerable amount of work has been done in estimating losses due to earthquakes [159-164].

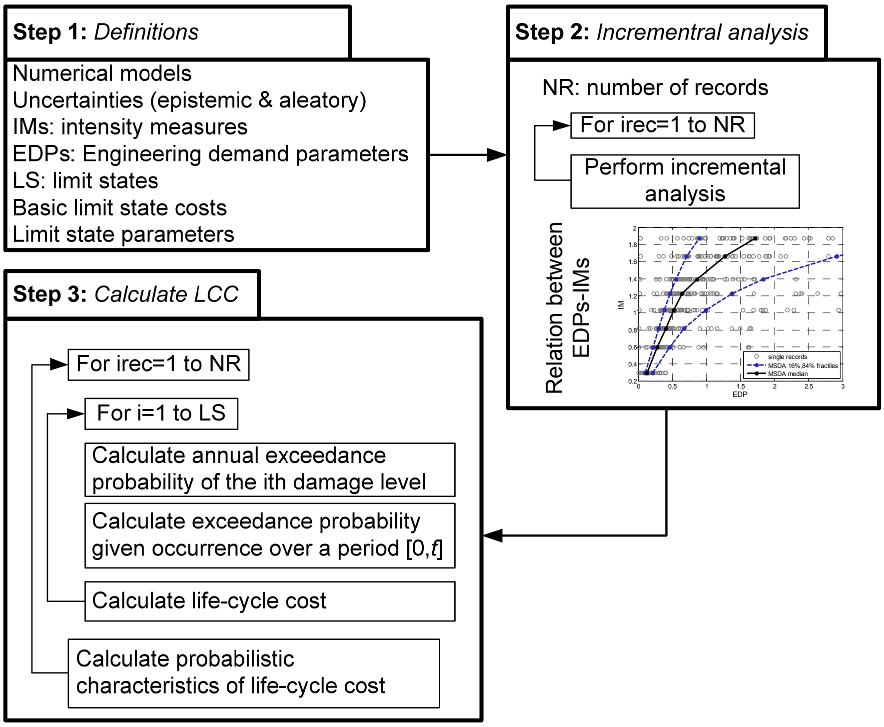


Figure 10 Flowchart of the life-cycle cost analysis framework.

In LCCA, the total cost C_{TOT} of a structure, may refer either to the design-life period of a new structure or to the remaining life period of an existing or a retrofitted structure. This cost can be expressed as a function of time and the design vector **s** as follows [165]:

$$C_{TOT}(t,\mathbf{s}) = C_{IN}(t,\mathbf{s}) + C_{LS}(t,\mathbf{s}) \tag{11}$$

where C_{IN} is the initial cost of a new or a retrofitted structure, C_{LS} is the present value of the limit-state (or life cycle) cost; **s** is the design vector corresponding to the design loads, resistance and material properties, while t denotes time. The term "initial cost" refers to the cost of a new structure or to the money invested for retrofitting an old structure. The term "limit-state cost" here refers to the potential damage cost from earthquakes that may occur during the life of the structure. It accounts for the cost of repairs after an earthquake, the cost of loss of contents, the cost of injury recovery or human fatality and other direct or indirect economic losses related to loss of contents, rental and income.

The steps of life-cycle cost analysis are shown in the flowchart of Figure 10. Damage is quantified with the aid of an engineering demand parameter (EDP) that is calibrated with respect to given damage states. So far a large number of researchers have studied various EDPs, while a detailed survey can be found in [166]. Damage, in the context of life cycle cost analysis, refers to both structural and non-structural damage. Peak interstorey drift (*IDR*) has been considered in the past as a suitable EDP. On the other hand, peak floor acceleration (*PFA*) is associated with the loss of contents like furniture and equipment [167,168]. Therefore, the limit-state cost (C_{LS}) of the i^{th} limit-state is [157]:

$$\begin{gathered} C_{LS}^{i,IDR} = C_{damage}^{i} + C_{contents}^{i,IDR} + C_{rental}^{i} + C_{income}^{i} + C_{injury}^{i} + C_{fatality}^{i} \\ C_{LS}^{i,PFA} = C_{contents}^{i,PFA} \end{gathered} \tag{12}$$

where the life-cycle cost is proportional to the limit-state costs of damage repair cost, loss of contents cost due to structural damage (quantified by the maximum interstorey drift), the loss of rental cost, income loss cost, cost of injuries and the cost of human fatality. $C_{contenrs}^{i,PFA}$ is the loss of contents cost due to peak floor acceleration. Details about the calculation formula for each type of limit-state cost along with the values of the basic cost for each category can be found in Table 2 [157,167].

Table 2 Limit-state costs - calculation formula [157].

Cost Category	Calculation Formula
Damage/repair (C_{dam})	Replacement cost × floor area × mean damage index
Loss of contents (C_{con})	Unit contents cost × floor area × mean damage index
Rental (C_{ren})	Rental rate × gross leasable area × loss of function
Income (C_{inc})	Rental rate × gross leasable area × down time
Minor Injury ($C_{inj,m}$)	Minor injury cost per person × floor area × occupancy rate × expected minor injury rate
Serious Injury ($C_{inj,s}$)	Serious injury cost per person × floor area × occupancy rate × expected serious injury rate
Human fatality (C_{fat})	Human fatality cost per person × floor area × occupancy rate × expected death rate

Based on a Poisson process model of the earthquake occurrences and the assumption that damaged buildings are immediately retrofitted to their original intact conditions after every major damage due to the seismic attack, Wen and Kang [165] proposed the formula for the limit-state cost considering *N* limit-states, while Mitropoulou *et al.* [167] added the *PFA* to account for losses of non-structural assets. The calculation formula becomes:

$$\begin{gathered} C_{LS} = C_{LS}^{i,IDR} + C_{LS}^{i,PFA} \\ C_{LS}^{i,IDR}\left(t,\mathbf{s}\right) = \frac{\nu}{\lambda}\left(1-e^{-\lambda t}\right)\sum_{i=1}^{N} C_{LS}^{i,IDR} P_i^{IDR} \\ C_{LS}^{i,PFA}\left(t,\mathbf{s}\right) = \frac{\nu}{\lambda}\left(1-e^{-\lambda t}\right)\sum_{i=1}^{N} C_{LS}^{i,PFA} P_i^{PFA} \end{gathered} \tag{13}$$

Where $C_{LS}^{i,EDP}$ is the damage cost for the ith limit-state violation calculated as described in Eqs. (12) for the two EDPs considered. The exponential term is used to express C_{LS} in present value and λ is the annual monetary discount rate [169]. The probabilities P_i^{EDP} (P_i^{IDR} , P_i^{PFA}) are calculated as follows:

$$P_i^{EDP} = P\left(EDP > EDP_i\right) - P\left(EDP > EDP_{i+1}\right) \tag{14}$$

where EDP_i, EDP_{i+1} are the lower and the upper bounds of the i^{th} limit-state and $P(EDP>EDP_i)$ is the exceedance probability given occurrence obtained as:

$$P\left(EDP>EDP_i\right)=-\frac{1}{vt}\ln\left(1-P_t\left(EDP>EDP_i\right)\right) \tag{15}$$

in which $P_t\left(EDP>EDP_i\right)$ is the exceedance probability over a period [0, t]; v is the annual earthquake occurrence rate modelled by a Poisson process and t is the service life of a new structure or the remaining life of a retrofitted structure. In order to calculate $P(EDP>EDP_i)$, we need to first find the t-year exceedance probability $P_t(EDP>EDP_i)$. Usually, t=1 year is considered and the annual exceedance probability $\overline{P}\left(EDP>EDP_i\right)$ is obtained using a relationship of the form [170]:

$$\overline{P}\left(EDP>EDP_i\right)=\gamma\left(EDP_i\right)^{-k} \tag{16}$$

Parameters γ and k are obtained by best fit of known P-EDP pairs, e.g. for events with 50, 10, and 2% probability of being exceeded.

6. Soft-computing design methods – Search Algorithms and Neural Networks

Traditionally, structural analysis methods were based on rigorous scientific procedures that are formed on mathematical methods and the principles of theoretical mechanics and led to the implementation of numerical simulation methods based on discretized continua. However, three decades ago new families of computational methods emerged and are known as soft computing (SC) methods. These methods are based on heuristic approaches rather than rigorous mathematics. Despite the fact that they were initially received with suspicion, many cases they turned out to be surprisingly powerful, while their use in various areas of engineering science is continuously growing. Artificial neural networks and nature inspired search algorithms are the most popular approaches of SC. Such algorithms have been implemented with success in earthquake engineering problems.

6.1. Nature inspired search algorithms and engineering optimization

Heuristic and metaheuristic algorithms are nature-inspired or bio-inspired as they have been developed based on the successful evolutionary behaviour of the natural systems by learning from nature. Nature has been solving various such problems over millions or even billions of years. Only the best and robust solutions remain based on the principle of the survival of the fittest. Similarly, heuristic algorithms use the trial-and-error, learning and adaptation to solve problems. Modern metaheuristic algorithms are almost guaranteed to an efficient performance for a wide range of combinatorial optimization problems. Loosely speaking, modern metaheuristic algorithms for engineering optimization include genetic algorithms (GA), simulated annealing, particle swarm optimization, ant colony, artificial bee colony, harmony search, firefly algorithm, and many others.

In structural optimum design problems the aim is to minimize an objective function under certain behavioural constraints. The objective function is often the cost of the structure or a quantity directly proportional to the cost. The constraints may refer to any engineering demand parameter (EDP) (e.g. stresses, stress resultants, displacements, maximum interstorey drift, plastic rotation, maximum floor accelerations, etc). A design $\mathbf{s}$ is a vector of design variables, $\mathbf{s}=[s_1, s_2, \ldots, s_{nd}]^T$, while the objective function is expressed as a linear or a nonlinear combination of $\mathbf{s}$. For skeletal structures, the design variables are usually chosen to be the cross-sections of the members of the structure. Due to engineering practice demands, the members are divided into groups of design variables, thus providing a trade-off between the use of more material and the need for symmetry and uniformity. Therefore, the number of design variables m may be less than the total number of beams and columns of the frame. Such problems are also known as discrete optimisation problems, since the design variables take values from a discrete set of values D^{n_d} such as tables of steel sections.

In the past a number of studies have been published where structural optimization with single and multiple objectives are solved implementing metaheuristics. Perez and Behdinan [171] presented the background and implementation of PSO suitable for constraint structural optimization problems. Hansen *et al.* [172] introduced an optimization approach based on an evolution strategy that incorporates multiple criteria by using nonlinear finite-element analyses for stability and a set of linear analyses for damage-tolerance evaluation. Kaveh and Shahrouzi [173] proposed a hybrid strategy combining indirect information share in ant systems with direct constructive genetic search. Farhat *et al.*[174] proposed a systematic methodology for determining the optimal cross-sectional areas of buckling restrained braces used for the seismic upgrading of structures against severe earthquakes. Chen and Chen [175] proposed modified evolution strategies for solving mixed-discrete optimization problems, in particular three approaches were proposed for handling discrete variables. Gholizadeh and Salajegheh [176] proposed a new metamodeling framework that reduces the computational burden of the structural optimization against the time history loading. Wang *et al.* [177] studied an optimal cost base isolation design or retrofit design method for bridges subject to transient earthquake loads. Hasançebi *et al.* [178] utilized metaheuristic techniques like genetic algorithms, simulated annealing, evolution strategies, particle swarm optimizer, tabu search, ant colony optimization and harmony search in order to develop seven optimum design algorithms for real size rigidly connected steel frames. Martínez *et al.* [179] described a methodology for the analysis and design of reinforced concrete tall bridge piers with hollow rectangular sections, which are typically used in deep valley bridge viaducts. Kripakaran *et al.* [180] presented computational approaches that can be implemented in a decision support system for the design of moment-resisting steel frames. Degertekin [181] proposed two improved harmony search algorithms for sizing optimization of truss structures, while four truss structure weight minimization problems were presented to demonstrate the robustness of the proposed algorithms. Comparative studies of metaheuristics on engineering optimization problems can be found in two recent studies by the authors [182,183] and in [184]. An overview of structural optimization problem formulation can be found in reference [185] and in the recent study by Fragiadakis and Lagaros [186].

6.1.1. Deterministic optimal design

A discrete deterministic-based structural optimisation (DBO) problem is formulated as:

$$\begin{array}{ll} \min_{\mathbf{s}\in F} & C(\mathbf{s}) \\ \text{subject to} & g_i(\mathbf{s}) \leq 0 \quad i=1,...,l \\ & s_j \in D^{n_d} \quad j=1,...,n_d \end{array} \tag{17}$$

where C is the objective function to be minimized, usually corresponding to the cost of the structural system that can be either the initial or the life-cycle cost of the structure. g_i are the l deterministic constraints and D^{n_d} is a given set of discrete values from which the design variables s_j, j=1,…, n_d take values. F is the feasible region where all the constraint functions g_i are satisfied. The above formulation is the simplest possible and is sufficient for a wide range of structural design problems.

6.1.2. Reliability-based optimal design

In real-world engineering uncertainties need to be taken into consideration during the design process. Uncertainties are always present and, depending on their source, they can be reduced but not avoided. A discrete reliability-based design optimisation (RBDO) problem introduces uncertainties as additional constraints [187-197]. Thus the problem formulation, takes the form:

$$\begin{array}{ll} \min_{\mathbf{s}\in F} & C(\mathbf{s}) \\ \text{subject to} & g_i(\mathbf{s}) \le 0,\ i=1,\dots,l \\ & \mathrm{h}^k\left(p_f^k(\mathbf{s}) \le p_f^{k,\lim}(\mathbf{s})\right),\ k=1,\dots,\mathrm{n} \\ & s_j \in \mathrm{D}^{n_d},\ j=1,\dots,n_d \end{array} \tag{18}$$

where h^k are n probabilistic constraints of the k^{th} limit-state. More specifically, $p_f^k(\mathbf{s})$ is the limit-state probability that has to be lower than a pre-set threshold $p_f^{k,\lim}(\mathbf{s})$. Alternatively, we may express the probabilistic constraints as function of the limit-state mean annual frequency (MAF) of exceedance. MAF convolves the limit-state probabilities $p_f^k(\mathbf{s})$ with the hazard at the site allowing setting design criteria on the annual frequency that the k^{th} limit-state is exceeded. The reciprocal of the MAF is the return period, in years, that the limit-state is exceeded and therefore it is easy to set meaningful thresholds. In both cases, the probabilistic constraints, h^k, are expressed in terms of an engineering demand parameter, used to identify the level of damage.

6.1.3. Robust optimal design

In robust design optimisation (RDO) an additional objective function related to the random nature of the problem parameters is considered. Therefore, in the case of a structural variant of RDO the aim is to minimize both the initial cost and the coefficient of variation (COV) of the response when the structural properties and/or seismic loading are considered as random variables. A thorough literature review on robust design optimisation can be found in [198-204]. An RDO problem is stated as a multi-objective optimisation problem as follows:

$$\begin{array}{ll} \min_{\mathbf{s}\in F} & [C_{in}(\mathbf{s}), COV_{EDP}(\mathbf{s},\mathbf{x})]^{\mathrm{T}} \\ \text{subject to} & g_i(\mathbf{s},\mathbf{x}) \le 0,\ i=1,\dots,l \\ & s_j \in \mathrm{D}^{n_d},\ j=1,\dots,n_d \end{array} \tag{19}$$

where $\boldsymbol{s}$ and $\boldsymbol{x}$ represent the design and the random variables vectors, respectively. The objective functions considered are the initial construction cost, C_{in}, and the coefficient of variation of an EDP, COV_{EDP}. In a multi-objective optimisation problem there is no unique point that would represent the optimum in terms of either minimum C_{in} and COV_{EDP} and thus the solution of the problem of Eq. (19) will provide the designer with a set of optimum solutions to select.

6.1.4. *Minimum life-cycle cost design*

Another formulation for seismic design would be the one that minimizes the total cost during the structure's lifetime, typically defined as the sum of the initial and the life-cycle cost. Life-cycle cost refers to the deterioration of the structural components' capacity over time due to phenomena such as corrosion or the deterioration of the joints or the bearings [205]. However, life-cycle cost may also refer to the risk related to natural hazards, such as wind or earthquake. In this case it is related to the possible losses due to the unsatisfactory performance of the structure under loading with random occurrence and intensity during its life. The design process should consider both direct and indirect economic and human life losses within a given social context [158].

The design for minimum life-cycle cost is preferably formulated as a two-objective problem and thus can be stated as:

$$\begin{aligned} &\min_{\mathbf{s}\in\mathrm{F}} && [C_{in}(\mathbf{s}), C_{lcc}(\mathbf{s})]^{\mathrm{T}} \\ &\text{subject to} && g_i(\mathbf{s}) \le 0,\ i=1,\ldots,l \\ & && s_j \in \mathrm{D}^{n_d},\ j=1,\ldots,n_d \end{aligned} \tag{20}$$

where C_{in} and C_{lcc} are the initial and the life-cycle cost objective functions, $\boldsymbol{s}$ represents the design vector that corresponds to the cross-sections of each member of the structure. Since this is a multi-objective problem, again a set of optimum designs are obtained.

6.2. **Neural Networks**

An artificial neural network is a massively parallel processor that can store experimental knowledge and make itself available for later use. It features adaptive learning, self-organizing capability during training and fault imprecision during applications. The main advantage of using Neural Networks (NN) is that they can deal with problems that do not have an algorithmic solution or for which an algorithmic solution is too complex to be found. Artificial neural networks are biologically inspired; they are composed by elements that perform in a manner analogous to the elementary functions of a biological neuron. These elements are known as artificial neurons. NN are organized in a way that is related to the anatomy of the brain and they exhibit a surprising number of the brain's characteristics such as: learning from experience, generalizing from previous examples and abstracting essential characteristics from sets of inputs containing irrelevant data. The use of hidden layers and nonlinear activation functions enhance the ability of the NN to "learn" the complicated relationship between a set of input and a set of output data.

Learning algorithms are classified to local or global algorithms. Global algorithms make use of the knowledge about the state of the entire network, such as the direction of the overall weight update vector. In the widely used back-propagation global learning algorithm the gradient descent algorithm is used. In contrast, local adaptation strategies are based on specific information of the weight values such as the temporal behaviour of the partial derivative of the weights. The local approach is better related to the NN concept of distributed processing, where the computations are performed independently. Moreover, it appears that for many applications local strategies achieve faster and more reliable predictions than global techniques. The Resilient backpropagation learning algorithm, abbreviated as Rprop [206], is a local algorithm, based on an adaptive version of the Manhattan-learning rule that has been proved very efficient in the past [207].

The response surface (RS) method was originally proposed by Box [208] as a statistical tool, to find the operating conditions of a chemical process at which some response was optimized. In order to reduce the computational effort while maintaining an acceptable accuracy, two important issues should be considered when applying the RS method to the failure probability: (i) The definition of experimental points for defining the approximation of the limit-state function, (ii) The analytical expression of the response surface function [209]. Usually, a quadratic RS function is assumed:

$$\overline{g}(\mathbf{x}) = a + \sum_{i=1}^{n} b_i x_i + \sum_{i=1}^{n} c_i x_i^2 \tag{21}$$

defined in an n-dimensional random variable space where the constants a, b_i and c_i are determined by evaluating $g(\mathbf{x})$ at certain specified experimental points, while x_i, i=1, …,n are the random variables.

RS method was found to be significantly influenced by the polynomial representation and the way the experimental points are generated. In an effort to improve the robustness of the approximation procedure the polynomial representation of RS is replaced by an approximation produced by neural networks. The global capabilities of the neural network approximator were examined [210,211] in the framework of FORM (First Order Reliability Method) for simple analytic limit-state functions or academic structural reliability problems. The local approximation capabilities of the neural networks in conjunction with reliability analysis methods are examined in a simple portal frame by Deng *et al.* [212]. Neural networks are implemented as an approximator of the limit-state function both for MCS and FORM [191,213,214]. In the first step a number of samples are generated through the Latin hypercube sampling (LHS) technique [215]. This sampling method can generate a variable number of samples well distributed over the entire range of interest. In the next step the sample points are used to define the NN approximation of the limit-state function according to the following equation:

$$\overline{g}(\mathbf{x}) = \mathsf{f}\left(\sum_{i=1}^{k} w_{H,ij} \cdot \mathsf{f}\left(\sum_{i=1}^{n} w_{O,ij} \cdot x_k\right)\right) \tag{22}$$

where k is the number of the hidden layer neurons, n is the number of the inputs for the NN that is equal to the number of the random variables, $\mathbf{w}_H$ and $\mathbf{w}_O$ are the weight parameters of the hidden and output layers, respectively, and f(y) is the sigmoid transfer function:

$$\mathsf{f}(y) = \frac{1}{1+\exp(-\gamma y)} \tag{23}$$

weight parameters are defined, MCS or FORM is performed in order to define the reliability index β.

7. Seismic assessment of lifelines

Modern societies rely heavily on civil infrastructures and utility networks for supporting day-to-day activities and for achieving socio-economic prosperity. Relevant networks such as the water, electricity, communication and transportation systems are often called lifelines; these networks are of primary importance of the well-being of communities. Unfortunately, life line elements and infrastructures are inherently complex, and as most man-made constructions, often prone to seismic events. As a result, extensive research has focused on the assessment of seismic risk on individual infrastructures (bridges, tunnels, water towers, dams etc) as a supporting tool for designing resilient network elements or retrofitting existing infrastructures. On the other hand, from a macroscopic, network (lifeline) perspective research has investigated the performance of lifelines as systems against seismic events at a lesser extent. However, from a planner and operator perspective, system-wide investigation of earthquake impacts on life lines may be critical for retrofit resource allocation and planning of post-seismic response and recovery actions. In that context, a review of the literature revealed 46 papers within the last 20 years, addressing the generalized problem of assessing the seismic performance and reliability of lifelines or focusing on particular cases of lifelines (transportation, electricity networks and so on).

Generalized approaches on seismic assessment of lifelines have been proposed by some researchers: Eguchi [216] examined seismic hazard input for lifelines and identified seismic hazard maps and their effects as important elements for relevant analyses. Two studies by Chen and Krauthammer [217,218] presented theoretical considerations and practical implementation related to modeling seismic effects on reinforced concrete lifelines. Selcuk and Yucemen [219,220] discussed seismic hazard lifeline performance; they identified three performance evaluation components, namely seismic hazard methodology, capacity determination and network reliability assessment and combined them into a single model for evaluating reliability of lifeline networks. Seismic vulnerability of lifelines was investigated by Menoni *et al.* [221]. The authors proposed a model for measuring response capacity of lifelines exposed to earthquakes. A systemic vulnerability concept was introduced by considering system failure as a result of damaging one of the system elements and the indirect effect of failures of other systems. The so-called RISK-UE methodology was presented by Pitilakis et al. [222] for assessing seismic risk of lifelines. The method considered a detailed inventory of lifeline elements along with their seismic hazard assessment, appropriate fragility models and models for estimating economic impact of lifeline damages. Recursive decomposition algorithms were developed by Li et al. [223] and Liu and Li [224], for examining the reliability of large networks against earthquakes. These models aimed at efficiently determining a network's disjoint minimal path. Menoni et al. [225] discussed frameworks for facilitating seismic vulnerability of lifelines in the context of interrelationships, stakeholders, information sources and data availability. Seismic assessment of interdependent lifelines was investigated by Kim et al. [226]. The authors developed a probabilistic model for characterizing the interdependencies between different lifelines and integrated it into a simulation model in order to estimate seismic performance of the overall system of lifelines. A review of seismic hazard analysis was offered by Klugel [227] from the perspective of potential users of related results. He identified shortcomings of probabilistic methods for describing seismic effects to lifelines and indicated that deterministic scenario methods provide a robust design basis but could be too conservative in some cases.

Javanbarg et al. [228] proposed a heuristic minimal path method for analyzing and evaluating seismic reliability of lifeline networks. Their method considered both link and node failures. The algorithm was used to construct the network's disjoint minimal path and disjoint minimal cut set and was applied to a number of different lifelines. Azevedo et al. [229] focused on summarizing and investigating interdependencies of the performance of different systems, including seismic input and output description and characterization, fragility of network elements and their performance. Interdependency related to system failure of multiple networks as a result of an earthquake was examined by Shoji and Tabata [230]. The authors used simulation in order to reveal network reliability and its variation for the nodes of network exposed to a seismic event. A GIS based software was developed by Selcuk-Kestel et al. [231] for depicting seismic reliability of lifelines; the software used a network reliability algorithm and presented upper and lower values of system reliability using seismic hazard and network information. Vulnerability of interdependent lifelines was again investigated by Wang et al. [232]; they proposed a framework for that purpose by considering both long-term and focused vulnerability and developed a method for ranking critical components of interdependent lifeline networks.

A number of authors investigate earthquake impacts to pipeline and urban water networks: Han and Sun [233]proposed a disjoint algorithm for analyzing the reliability of a buried pipeline network and for constructing its disjoint minimal path. A model for predicting earthquake damage to a waterway network was developed by Toprak et al. [234]. Adachi and Ellingwood [235] analyzed the effects of power availability to water systems due to earthquakes using a fault tree analysis and a shortest path algorithm. Maugeri et al. [236] applied models that evaluated seismic hazards and expected damage on a pipeline network and incorporated them in a GIS environment. A general review on risk mitigation and vulnerabilities of pipelines with respect to earthquakes can be found at [237]. The author also presented failure modes of over and underground pipeline networks. Seismic risk affecting oil and gas pipelines in the Republic of Georgia were identified and quantified by Pasquare et al. [238]. Recently, Zohra et al. [239] developed a method for assessing vulnerability of water supply networks, based on the identification of those parameters affecting pipeline behaviour to seismic events.

Some studies focus on electricity networks; Ang et al. [240] and Pires et al. [241] proposed models for probabilistically assessing structural damage and abnormal flow at an electricity network for a given seismic event. Vanzi [242] developed a reliability model for electricity networks; his model assembled small into larger network elements and considered modified power flow equations in order to take into account seismic conditions. Rose et al. [243] investigated regional economic impacts of electricity disruptions due to an earthquake using input-output and linear programming models. An application of their model for Memphis, Tennessee revealed a 7% reduction in the regional GDP due to power losses resulting from a seismic event. Cagnan and Davidson [244] proposed a simulation-based methodology for modeling post-earthquake restoration actions for an electric power system. A software package (Computational Intelligence Applications to Power Systems), incorporating metaheuristics for assessing vulnerability of electric networks was developed by Haidar et al. [245].

In the area of highway and transportation networks, over twenty papers on seismic risk assessment have been published in the last twenty years. Nicholson and Du [246] developed a network equilibrium model describing a transportation network affected by a earthquake, with degrading links. The authors provided a tool for identifying socioeconomic impacts of a network's degradation along with an analysis of critical components and reliability. Post-disaster performance measures for transportation infrastructures were developed by Chang and Nojima [247]; using these measures they evaluated the impacts of the 1995 Kobe earthquake. Pipeline vulnerability knowledge was exploited by Torres-Vera and Canas [248] for examining vulnerability of Barcelona's subway following a disaster. Loh et al. [249] used a limited number of free-filed ground motion data in order to evaluate the condition of a transportation network immediately after an earthquake. Murray-Tuite and Mahmassani [250] used a non-zero sum game for modeling the vulnerability of a transportation network's links. Ham et al. [251] developed an input-output model for assessing impacts of a disaster to a multi-commodity, multi-modal transportation network. Latora and Marchiori [252] presented a general method for identifying the critical components of an infrastructure network and Jenelius et al. [253] developed measures for assessing the vulnerability and reliability of transportation network links by considering the generalized transportation cost when links are closed. Seismic risk assessment of transportation networks as a result of direct damage loss to bridges was investigated by Kiremidjian et al. [254], while Padgett and Desroches [255] formed a seismic risk assessment framework of a highway system by considering fragility and functionality of its elements. Ukkusuri and Yushimoto [256] proposed a heuristic procedure based on complex network science to assess the importance of highway transportation networks using travel time as the performance measure. Earthquake insurance rates for the case of a motorway were probabilistically assessed by Yucemen et al. [257]. Uncertainty in the risk of seismic assessment in highways was the topic of a paper by Banerjee and Shinozuka [258]. The authors combined earthquake scenarios, simulation for estimating infrastructure damage and a traffic assignment model for assessing network performance and developed seismic risk curves showing probabilities of exceeding social costs of networks. Based on these curves they quantified uncertainties in the different elements of risk assessment. Bocchini and Frangopol [259] developed a computational method for jointly analyzing the fragility of transportation networks and traffic flow distribution, under extreme events. On a different perspective, Anbazhagan et al. [260] proposed a novel scale for classifying and quantifying earthquake damage of highway networks, based on collected data from past seismic events. A practical method for analyzing reinforcement of transportation networks against earthquakes and their associated risks was developed by Nagae et al. [261]; the method estimates the performance of reinforcement strategies and estimates damage patterns of highway networks and their occurrence probabilities based on a clear, straightforward logic.

8. Conclusions

A wide discussion on the seismic assessment of structures and lifelines has been presented. The paper provides a critical overview on the current state-of-the-art and discusses the most popular approaches for analysis, modeling and design of structures prone to seismic actions. It is clear that earthquake engineering is a multi-disciplinary field that requires a deep knowledge and understanding of interrelated topics in order to efficiently model structural response under extreme seismic loading. Emphasis has been given to the treatment of uncertainty, due to the nature of seismic loading and also thanks to the significant advances in modeling seismic actions and the development of novel probabilistic performance estimation methods. Moreover, it is well understood that assessing the seismic risk of lifelines is essential for post-earthquake resilience and thus recent advances in this field of earthquake engineering are also discussed. In all, as our understanding on earthquakes is improving, engineers need to combine sophisticated computational tools and earthquake engineering performance estimation methods in order to mitigate seismic risk.

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