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Trends and Challenges of Optimal Performance-based Earthquake Resistant Design Criteria for Buildings

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International Association for Earthquake Engineering

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by

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Curriculum Vitae of Luis Esteva Maraboto

Education

BS, Civil Engineering, National Autonomous University of Mexico, Mexico City, Mexico	1958
MS, Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, USA	1959
Ph.D., Civil Engineering, National Autonomous University of Mexico, Mexico City, Mexico	1968

Professional Appointments (selected)

Structural design, practical and consulting activities	1955-2021
Participant in the formulation of seismic design codes in Mexico and several countries in Latin America	1959-2021
Professor and Researcher in the School of Engineering and the Institute of Engineering of the National Autonomous University of Mexico	1959-2021
Visiting Professor, Massachusetts Institute of Technology	1969
Visiting Professor, University of Innsbruck	1988
Director, Institute of Engineering, National University of Mexico	1982-1991
Coordinator of Scientific Research, National University of Mexico	1991-1993
Visiting Professor, Stanford University, USA	2001-2002
President, International Association for Earthquake Engineering	2002-2006

Honors and Awards (selected)

National Science Award, Mexican National Academy of Science	1970
Professor Honoris Causa, Catholic University of Argentina	1980
Guest visitor of the Academy of Science of the Soviet Union	1980
National Award in Science and Technology, Mexico	1981
Member of the Mexican Academy of Engineering	1982
Technology Award of TWNSO (Third World Network of Scientific Organizations)	1993
Corresponding Member, National Academy of Engineering of Argentina	1996
Corresponding Member, Organization of Scientists of the Region of	

Primorie, Russia	1997
Honorary Member, International Association for Earthquake Engineering	1998
Foreign Member, US National Academy of Engineering	2000
G. Housner Medal, Earthquake Engineering Research Institute	2005
National Award of Civil Protection, Mexico	2005
Honorary Member, International Association for Life-Cycle Civil Engineering	2012
National Award of Civil Engineering, Mexico	2016

PREFACE

On behalf of International Association for Earthquake Engineering (IAEE), I am very pleased to announce that IAEE has launched a new initiative named "Masters Series" in 2018. The objective of this initiative is to connect the legends in our discipline of earthquake engineering with those who shall lead our discipline now and in the future. The initiative consists of three categories, namely "Read the Masters", "Meet the Masters", and "Greet the Masters". Among these, "Read the Masters" is for a legend to write a monograph on the subject of his or her expertise and share the legend's efforts and experiences with the next generations. The other two, "Meet the Masters" and "Greet the Masters" are the programs that will connect our legends with the next generations during the World Conference on Earthquake Engineering, which is to be held once every four years in various parts of the world.

The very first product of the monograph, "Read the Masters", is written by Professor Luis Esteva Maraboto, of the National Autonomous University of Mexico (Universidad Nacional Autónoma de México). He has led the research on seismic hazard and risk analysis, life-cycle optimization in earthquake engineering, and seismic vulnerability analysis of irregular buildings, among others. He also served as President of IAEE in 2002 to 2006.

I hereby wish the readers to enjoy the reading on the accomplishment of a great master of earthquake engineering.

Masayoshi Nakashima
President of IAEE (2018-2022)

Contents

1 INTRODUCTION AND SCOPE

Performance-based criteria for earthquake resistant design intend to comply with two simultaneous conditions: obtaining optimum utility solutions in the frame of a life-cycle context, while maintaining expected damage and risk levels within values accepted by society. This requires the development of quantitative probabilistic models of seismic hazard at the location of the system of interest, as well as of the vulnerability function of that system, including its evolution resulting from the influence of damage accumulation and the repair and replacement strategies adopted. The vulnerability function depends on the design criterion adopted. The evaluation of both functions, seismic hazard and system vulnerability, must take into account the significant uncertainties that affect them. They must include those related to the stochastic models of seismic activity, the intensity attenuation functions in terms of magnitude and site-to-source distance, the local soil properties, the gravitational loads and the mechanical properties of the systems considered.

The following sections present a summary of the challenges faced in the formulation of performance-based earthquake resistant design criteria satisfying the expected performance targets mentioned above, while maintaining a simple format, adequate for modern engineering practice

2 SEISMIC PERFORMANCE OF BUILDINGS: LESSONS LEARNED

Significant advances have taken place, during the last decades, in the basic approaches, structural system options, dynamic-response control and reduction systems and computational tools applicable in the practice of Earthquake Engineering. However, seismic-induced disasters continue to occur in important urban areas throughout the world. This is true even for regions that count with modern technology and wide economic resources. In some cases, unforeseen excessive damage levels can result from excessively lax or insufficiently conservative design and construction practices. These attitudes often arise from the underestimation of seismic hazard by the engineering community or by the normative groups; the latter often base their hazard estimates on very small statistical samples of the activity of potential seismic sources near a site. Lack of consciousness or of information about the possible influence of local soil conditions is often an additional element in a chain of concepts that contribute to enhance seismic hazard and risk.

A significant portion of the damage produced by earthquakes can be associated with old constructions; many of them non-engineered, others built in accordance with obsolete or inadequately applied building codes and norms. However, observations about the performance of constructions during recent earthquakes around the world have disclosed previously ignored sources of higher than expected levels of seismic hazard and structural vulnerability, thus calling the attention about previously unnoticed risk-enhancing concepts. This is true even for structures designed and built in accordance with state-of-the-knowledge seismic design norms.

For many years, the practice of earthquake-resistant design has concentrated on the construction of systems capable of resisting large lateral forces without suffering collapse, but it has largely ignored the need to control direct and indirect costs of structural and non-structural damage for the condition of system survival. Conscience about this need has existed only during the last few years. On the other hand, for a long time, deformation and energy-dissipation capacities have been essential concepts for the prevention of system collapse during high intensity earthquakes. However, only during the last few years ordinary structural design criteria and quality control regulations have introduced detailed explicit requirements oriented to ensuring, with a sufficiently high probability, the dominance of ductile failure modes and the prevention of brittle failure modes for the system of interest as a whole. Modern performance-based seismic design criteria openly address the need to produce a structural system capable of developing the required deformation- and energy-dissipation capacities, consistent with its shear strength and with the corresponding expected dynamic response demands. As shown later, the fact that previous generations of seismic design norms do not explicitly recognize the significance of these concepts can help to explain many of the observed cases of severe damage or collapse.

The lessons learned from observations about the seismic response and behavior of urban constructions have fostered the development of a new generation of seismic design recommendations and codes, which focus attention on system performance rather than on lateral strength or stiffness. This book intends to describe those lessons, which are common to seismic events around the world, and to present an overview of the most significant trends and challenges they have fostered in the general approach to earthquake resistant design of urban constructions. This approach recognizes that a specified target level of expected performance must be expressed as corresponding to a seismic event with an intensity defined in terms of a given probability of occurrence during a given time interval. For this reason, detailed quantitative information must be included, in order to characterize seismic hazard at a site of interest, for instance, by means of uniform-hazard response spectra. Given a uniform-hazard response spectrum as the basis for design, it is necessary to develop practically applicable criteria and algorithms, capable of producing reasonably accurate estimates of the expected values of the maximum response amplitudes and of the uncertainties affecting them.

The concepts described above generate the need to examine a wide variety of concepts, ranging from the probabilistic models used to represent the seismic excitations to the computer tools needed to estimate the dynamic responses of nonlinear systems subjected to high intensity excitations and the calibration of the simplified criteria and models adequate for application in usual engineering design practice. In accordance with this, the following sections intend to present a brief description of the following essential concepts:

a) Seismic-hazard assessment,
b) Refined and approximate methods for the estimation of seismic vulnerability functions for single- and multi-component seismic excitations, including the influence of stiffness and strength in-plan eccentricities,
c) Influence of irregularities along the height of the system,
d) Dual systems with frames and shear-walls,
e) Influence of initial damage,
f) Non-symmetric shear-distortion functions,
g) Actions to ensure ductile system behavior,
h) Damage-location control,
i) Life-cycle optimization,
j) Passing from research results to practically applicable design criteria

and algorithms.

The rest of this chapter presents a brief description of some of the main concepts that have contributed to the generation of significant damage in urban constructions during the last decades.

2.1 Seismic hazard assessment: information and models

The estimation of seismic hazard at a given site usually makes use of different types of direct or indirect information, which includes several concepts, such as the following:

a) Observed seismic intensities,
b) Performance of existing constructions,
c) Instrumental ground motion records or mathematical models of the activity of the potential seismic sources near the site and of the intensity attenuation functions relating adequate engineering intensity measures with the magnitude and the site-to-source distance.

In many cases, the available information may be either inadequate or insufficient and, therefore, misleading. The shortness of the time interval covered by that information is usually a significant source of uncertainty affecting the quantitative estimates of seismic hazard. The following paragraphs describe a few experiences related to the underestimation of this variable.

- *Insufficient historic evidence*

Take for instance the Michoacán earthquake of September 19, 1985, which produced significant human and economic losses in Mexico City, lying 360km from the epicenter. The city includes one of the largest urban areas in the world; unfortunately, one with the most difficult foundation conditions. A large portion of the constructions in the city lie on soft clay sediments, with depths that may be as large as 70m, with shear wave velocities as low as 90m/s at some sites, and dominant ground periods as high as 4.5 in the Eastern side of the city or 2.0-2.5s in the downtown area. In spite of its long history and its unfavorable local soil conditions, the city had never experienced an earthquake as damaging as that of 1985. This is easy to explain in terms of the types of constructions that were exposed to earthquakes during the last six centuries, before the advent of modern high-rise constructions, in the middle of the twentieth century. As mentioned above, they were short-period constructions, not sensitive to the narrow-band ground motions with energy inputs concentrated in dominant frequencies corresponding to the dominant ground periods. Prior to 1985, the highest damage ratio ever recorded throughout the urban area was that produced by the earthquake of July 28, 1957, with a magnitude $M_S = 7.6$ and a source-to-site distance of 256km, which resulted in the collapse of a few buildings and the loss of about 100 lives.

The evolution of seismic design regulations in the city during the interval 1957-1985 largely reflected the experiences about structural performance during the 1957 event and during a number of moderate intensity shocks occurring every five years in the average. The highest acceleration observed at a soft-soil site during this interval was estimated as 0.06g; that is, about one third of that recorded in 1985 near the SCT building, within the area of the city with the highest damage level (Esteva, 1988).

- *Imperfect seismic-tectonic knowledge and models*

The Northridge earthquake of January 17, 1994 occurred on a deep thrust ramp beneath the San Fernando Valley, California. It had a magnitude $M_S = 6.8$. It constituted a surprise for seismologists, because it did not match with the information that was then available about superficial geological features. However, its occurrence is consistent with the high activity rate in the Los Angeles area during the twenty years preceding it, thus emphasizing the seismic hazard associated with concealed faults, a concept recognized among the geophysicists since the 1987 Whittier Narrows earthquake (Hauksson & Jones, 1995). Two examples of ground-motion time-histories of this event are presented in Figures 1 and 2. The corresponding response spectra are shown in Figures 3 and 4; they are compared with the design spectra specified in the UBC Building Code valid at that time.

The Great Hanshin (Kobe) earthquake of January 17, 1995 has been the most damaging seismic event in Japan since the Great Kanto Earthquake of 1923. It had a magnitude equal to 7.2, and it resulted from the rupture of a shallow-focus event generated at a strike-slip fault lying directly under downtown Kobe. The contribution of an event like this to seismic hazard in the area had been ignored, because of an extended belief that seismic hazard was essentially determined by the deeper-focus earthquakes generated at the fault zone where the Pacific Plate is sub-ducting under the Philippine sea plate.

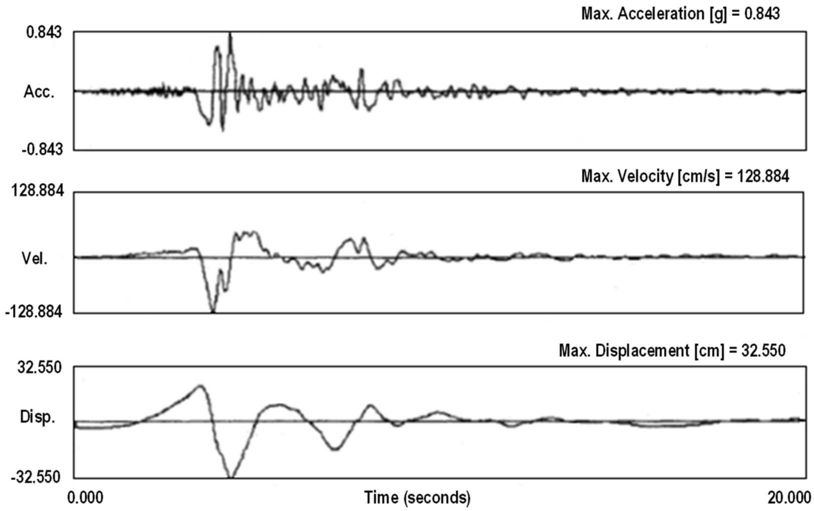


Figure 1. Ground motion time histories at Sylmar County Hospital Parking Lot (N-S) (Northridge, 1994; Naeim, 1995)

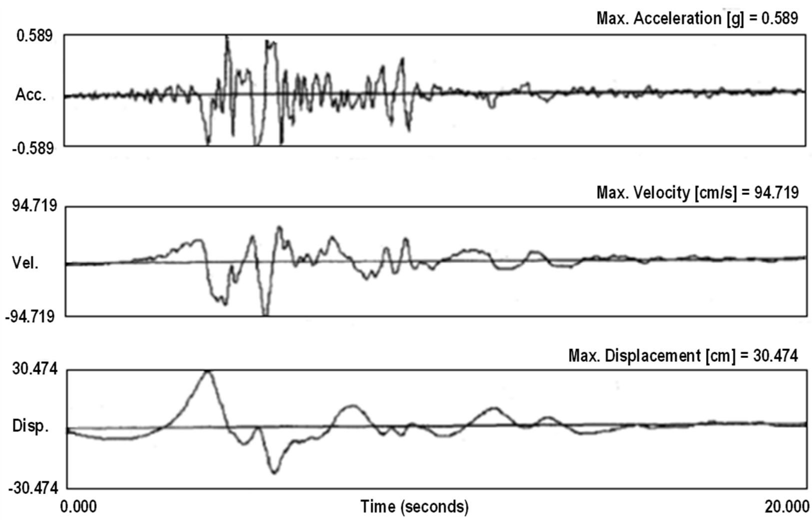


Figure 2. Ground motion time histories at Nehall County Fire Station (N-S) (Northridge, 1994) (Naeim, 1995)

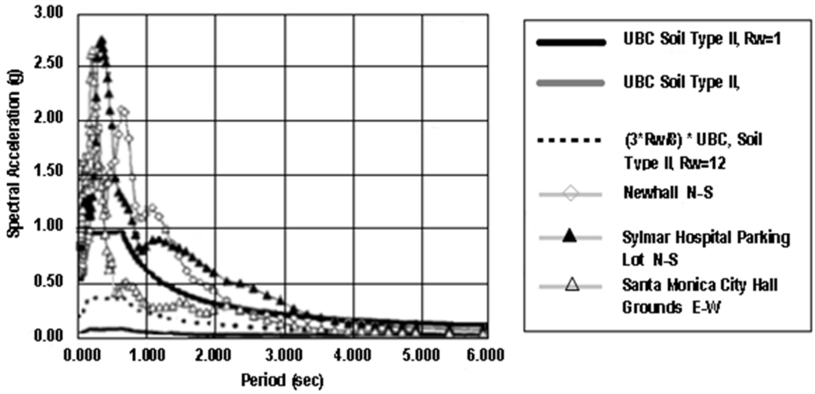


Figure 3. Elastic response spectra for selected Northridge 1994 records and UBC design spectra

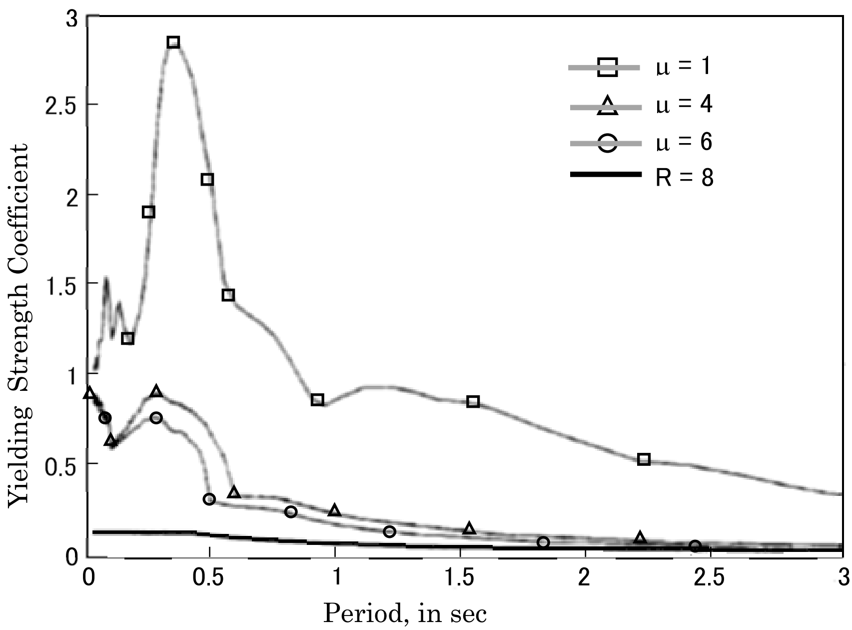


Figure 4. Inelastic response spectra (5 % damping) for the Sylmar Parking lot, compared with code design spectrum for an $R_w = 8$ system (Northridge, 1994; Naeim, 1995)

The Nisqually earthquake of February 28, 2001 was one of the largest recorded earthquakes in a zone along the Western coast of North America, known as Cascadia, going from the North of California to the border between the United States and Canada. It was an intra-slab earthquake, with a magnitude $M_S = 6.8$; its epicenter was under Anderson Island, about 17 km Northeast of Olympia, with a focal depth of 52 km. This event caused some property damage in Seattle and surrounding areas. The zone of Cascadia has been associated with one of the world's quietest subduction zones, only recently recognized as one of high seismic hazard, considering the strain-energy accumulation process due to the under-thrusting of the Juan de Fuca and Punta Gorda oceanic plates beneath North America, which has been taking place without frequent release of energy. This has generated increasing concern about the possibility of occurrence of a large earthquake (M > 8) in this region (Atwater *et al*, 1995).

- *Non-applicable intensity attenuation functions*

Using the information derived from strong-motion instruments placed on firm ground at sites lying between Mexico City and the southern coast of the country, Ordaz and Singh (1992) obtained attenuation equations for ordinates of the Fourier amplitude spectra of the ground acceleration. They show evidences suggesting the occurrence of significant amplification of those ordinates on firm ground sites within the vicinity of Mexico City, with respect to those estimated by means of the attenuation equations established from the information of records obtained at the other sites. An example of this is shown in Figure 5 for a frequency equal to 0.5 hertz.

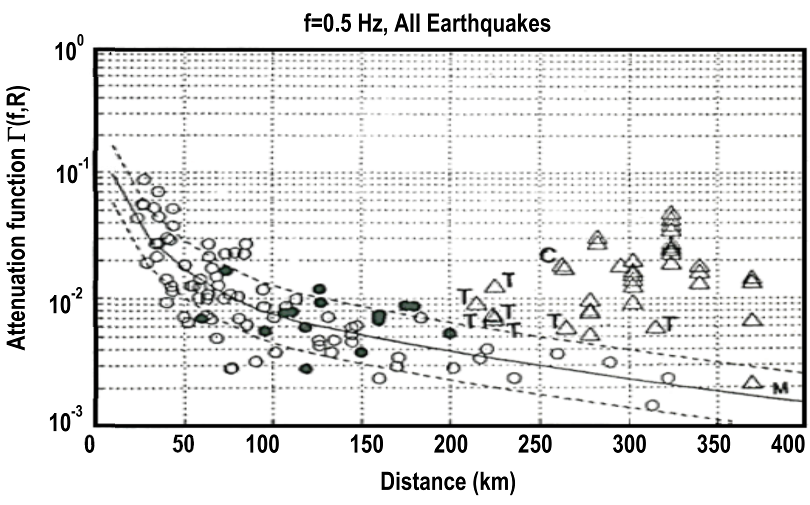


Figure 5. Evidence of amplification of earthquake spectra on the surface of volcanic rock formations (Mexico City, 1985; Ordaz and Singh, 1992)

A careful analysis of this information led to the conclusion that even sites known as “firm ground” in the Valley of Mexico are subjected to significant local amplifications of ground motion, due to the low values of S waves that characterize the volcanic rocks that underlie the valley. This is an important concept to have in mind for the purpose of seismic hazard assessment.

- *Near-source ground motion time histories and response spectra*

A large number of ground acceleration records were obtained during the Northridge earthquake of January 17, 1994. The information provided by them has called the attention to several highly relevant concepts related to the near-source characteristics of ground motion during high intensity seismic events and their implications on the formulation of practical seismic design criteria with consistent safety levels. If compared with typical acceleration records on firm ground, near-source records ordinarily show longer dominant periods, concentrating the energy input on a small number of acceleration or velocity pulses. As a result, the maximum ordinates of the linear pseudo-acceleration response spectra appear at natural periods longer than the typical ones of ground motion records on firm ground at short and moderate distances. Peak ground accelerations were equal to 0.88g and 0.86g at Santa Monica City Hall Grounds and Sylmar County Hospital Parking lot, respectively, but they did not reach the values ranging from 0.90g to 1.50g recorded during previous seismic events in the Western coast of the United States. However, the peak ground velocity of 128.9cm/s recorded at the second site mentioned above was the largest one recorded previously (Naeim, 1995). Another important characteristic of the ensemble of records obtained is the large statistical dispersion of the peak ground accelerations, which is a consequence of the fact that the distance to the epicenter is not representative of the closest distance to the zone of energy release. A significant element in this dispersion is the contribution of the so called “*directivity effect”*, which manifests itself by systematic differences in the duration and the frequency content of the ground motion records at different sites, depending on their location with respect to the direction of propagation of the seismic ruptures along the fault causing the seismic event. Elastic pseudo-acceleration response spectra resulting from the records mentioned in the last few lines were significantly higher than the spectra specified in the UBC design requirements in force at the time of the earthquake, particularly for natural periods shorter than 1.0s. An examination of the elastoplastic response spectra for different ductility levels shows that the equal-displacement rule, normally accepted to estimate the ordinates of elastoplastic response spectra for moderate and long natural periods, will lead to non-conservative values of the ductility-based reduction factors of pseudo-acceleration spectral ordinates (Figures 1-4, taken from Naeim, 1995). Energy spectra for the Northridge records mentioned above show ordinates 50 percent higher than those observed in any record from previous events for natural periods ranging from 1.0 to 3.0s. These features have significant implications on the required seismic design criteria

2.2 Seismic vulnerability of existing structural systems

A large percentage of the cases of engineered urban constructions affected by severe damage, partial or total collapse during earthquakes, are associated with faulty engineering practice, either during the design process or during the construction and quality control activities. In some cases, evidence of non-compliance with standing building code requirements is obvious; in other constructions, assumed to have been designed and built under the guidance of those requirements, some design mistakes arise from failure to interpret them or from conscious neglect of details wrongly taken by the designer as irrelevant, too complicated or too expensive to implement. In most cases, those details are specified in order to prevent brittle failure, either at a local or at a global level. More often than desirable, applicable seismic design regulations and generally accepted design and construction guidelines are deficient, outdated or established from the information resulting from an event with a ground-motion intensity lower than that of the seismic event considered of interest.

Observations about the performance of urban constructions during moderate- and high-intensity earthquakes have taught many lessons, disclosing or stressing the influence on that of the mistakes and deficiencies mentioned in the foregoing paragraphs. Some illustrations about them are presented in the following; most of the material presented corresponds to observations about the seismic performance of constructions in Mexico, because it was readily available to the author. However, it is representative of typical observations made at many different sites around the world. For simplicity, a parenthesis has been inserted in the title of each picture used to illustrate the occurrence (or non-occurrence) of a given damage or failure mode, in order to identify the seismic event associated with the performance evidence shown.

For the case of Mexico City, 1985, the following information is essential for a better understanding of the possible causes of the observed performance: When the 1985 earthquake occurred, existing seismic design codes were essentially the result of the experience derived from the 1957 earthquake. The urban area was divided into three micro-zones: I Firm ground, II Transition zone (soft sediments with a depth lower than 20m) and III Soft soil (soft sediments deeper than 20m). Linear response spectra (5 percent damping) specified for design had maximum ordinates of 0.16, 0.20 and 0.24 for these zones, respectively. Reduction factors to account for nonlinear behavior ranged from 1 to 6, depending on the type of structural arrangement and the ductility-oriented details (García-Ranz and Gómez, 1988). These values are equal to the ordinates predicted by means of the attenuation functions derived from the information obtained from previous seismic events, which had greatly influenced decisions about codified seismic design requirements; however, they were widely exceeded by those resulting from the ground motion records obtained during the 1985 earthquake. Figure 6, taken from Castro *et al* (1988), shows this.

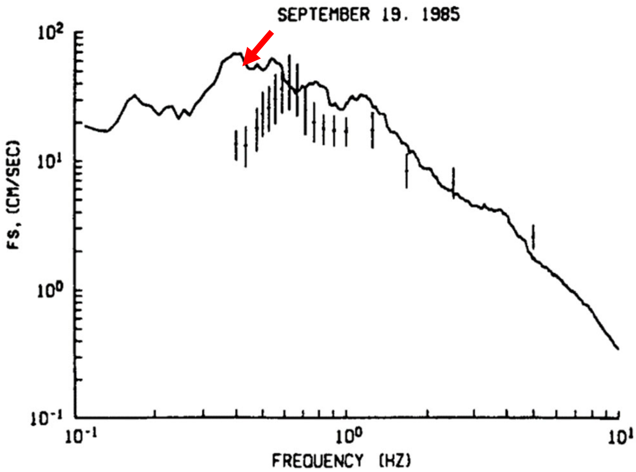


Figure 6. Comparison of observed and predicted values of the Fourier amplitude spectrum of the EW-SCT-850919 ground motion acceleration record (Mexico City, 1985; Castro *et al*, 1988)

Given the overwhelming differences between observed intensities and design capacities, it must be concluded that the stability of constructions rested mainly on the reserves associated with the strength of elements not accounted for in design, as well as with the energy dissipating capacity of those elements and of those that were recognized as structural members. Therefore, with the aim of improving the technical basis for earthquake engineering, understanding the survival of so many structures during this earthquake (and during many others, for this matter) is at least as important as identifying causes for the failures observed.

- *Pronounced asymmetry in stiffness*

Figure 7 shows the post-earthquake configuration of a building with significant in-plan strength and stiffness eccentricities. The residual displacements were not acceptable, both from esthetical and from enhanced risk considerations; therefore, the building had to be demolished.

Figure 8 shows a sketch of a plan of a typical "corner building" (located at the corner of a city block) in Mexico City. It contains infill masonry walls along edge-frames away from the street and simple frames along the building facades facing the two streets at the corner. This configuration gives place to very high eccentricities in lateral stiffness and strength, which were responsible of 42 percent of the number of buildings that experienced severe structural damage or collapse in Mexico City during the 1985 Michoacán earthquake.

Figure 7. Pronounced asymmetry of stiffness (Mexico City, 1985)

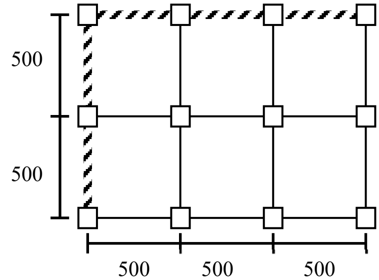


Figure 8.
Corner building, with infill masonry walls in edge-frames away from the street facades

- *Weak first story*

Figure 9 shows a typical weak-first-story building in Mexico City. This particular system remained standing, but its performance cannot be taken as representative of that of many others, similar to it, which have suffered collapse during seismic events around the world (Figure 10). On-site observations, and theoretical considerations as well, have shown the extremely high seismic vulnerability of systems with significant along-height strength and stiffness variations; this arises from the high concentrations of ductility demands that occur at a weak story. The latter is a story with a significantly lower ratio of linear dynamic story shear to lateral shear capacity than the average value of this ratio along the building height. Figure 11 shows one of many cases where failure of this type was observed during the L'Aquila (Italy) earthquake of 2009 (Celebi *et al*, 2010). As shown later, the probability of occurrence of these ductility-demand concentrations is very sensitive to the ratio of the fundamental period of the building to the dominant frequency of the seismic excitation.

Figure 9. Typical weak-ground-story building (Mexico City, 1985)

Figure 10. Collapse of weak ground story of building, with survival of upper stories (Gujarat, 2001; Murty *et al*, 2002)

Figure 11. Along-height irregular building: weak ground story (L'Acquila, 2009; Celebi *et al*, 2010)

To a lesser extent, large concentrations of ductility demands may also occur in irregular buildings, with flexible floor diaphragms and non-uniform distributions of the ratio of linear dynamic story shear to lateral shear capacity along the building length, as shown in Figure 12 (Murty *et al*, 2002). The compression failure of a column shown in Figure 13 corresponds to a different mechanism: here, the presence of an elevator core built with reinforced concrete shear walls prevented the occurrence of large lateral story distortions. The failure in compression illustrates a case where this condition, dominated by a non-ductile mechanism, is easy to prevent by means of a design criterion that explicitly applies a higher safety factor to the observed failure mode than to that associated with ductile distortions of the upper stories.

Figure 12. Partial collapse of one end of a long building with flexible floor diaphragms (Gujarat, 2001; Murty *et al*, 2002)

Figure 13. Weak-ground-story building with elevator core; compression failure of column due to overturning moment (Mexico City, 1985)

- *Masonry infill elements*

Masonry diaphragms often play the role of structural or infill elements in reinforced concrete or structural steel rigid frames. Due to their inability to develop ductile behavior, they often experience severe damage, as shown in Figures 14 a) and b); however, they can provide significant contribution to the energy dissipation capacity of a building, and therefore help to improve its seismic performance. In order to profit from this capacity, it is necessary to confine the masonry diaphragms by beams and columns with sufficient shear capacity at their ends to support the forces transmitted by those diaphragms, acting as compression diagonals. Figure 12 shows a case where the masonry diaphragms in the transverse direction of a long building prevented the collapse of a large portion of it. The collapse of the end of the building occurred because of the lack of lateral strength of the rigid frames located at those ends, coupled with the incapacity of the flexible floor diaphragms to transmit to the masonry diaphragms the inertia forces acting in the short direction of the building plan.

- *Short columns*

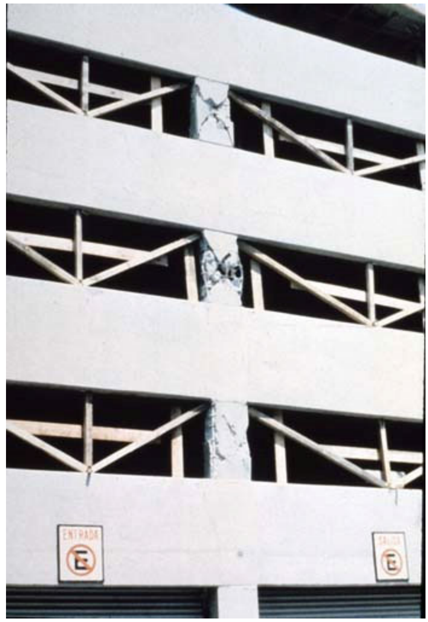
Unless designed with criteria explicitly aimed at preventing the occurrence of a diagonal-tension failure mechanism before one associated with ductile bending failure at the member ends, short columns (small length/depth ratio) will have a large probability of suffering a brittle failure, as shown in Figure 15.

a) Infill elements (Mexico City, 1985)

b) Structural shear walls (Gujarat, 2001)

Figure 14. Shear failure of masonry–elements

a) Concepción, Chile, 2010
(Elnashai *et al*, 2010)



b) Mexico City, 1985

c) Bingöl, Turkey, 2003; the ground story is completely collapsed
(Gur *et al*, 2009)

Figure 15. Brittle failure of short columns

- *Excessive gravitational loads (change of use)*

Figure 16 shows evidence of the application of live loads significantly larger than those assumed for structural design. This condition, associated with change of use of the system, prevailed in 9 percent of the cases of excessive damage or collapse in Mexico City in 1985 (Meli & Rosenblueth, 1986).

- *Impact between adjacent buildings*

An intermediate story of the building shown in Figure 17 collapsed, due to the impact of the building with another adjacent to it. This type of failure, which occurred in 15 percent of the cases of significant damage or collapse in Mexico City in 1985, could have been avoided by leaving an adequate space between both systems.

Figure 16. Excessive gravitational loads associated with change of use of building (Mexico City, 1985)

Figure 17. Impact between adjacent buildings (Mexico City, 1985)

- *Foundation failure*

Figure 18 shows several examples of foundation failure occurring in Mexico City in 1985:

a) Excessive differential settlements (1.6m) and tilting (6 percent) of a building. A settlement of 0.58 cm had already occurred prior to the earthquake.
b) Significant residual tilting associated with differential settlements; at least a part of them had occurred before the earthquake. The structure had to be demolished.

c) Overturning failure of a building, resulting from bond failure of friction piles.

a) Foundation bearing failure

b) Excessive tilting

c) Bond failure of friction piles

Figure 18. Foundation failure (Mexico City, 1985)

In all these cases, failure might have been avoided through the application of a criterion ensuring higher safety factors for the mentioned failure modes than for those associated with the development of the lateral shear capacity of the super-structure.

- *Punching in waffle slabs*

The waffle-slab structure shown under construction in the left portion of Figure 19 is typical of those used for moderate height (10-15 stories) buildings in Mexico City, prior to the 1985 earthquake. Very few of them were capable of resisting the lateral force demands generated by the ground motion. The weakness of the slab-to-column connections was clearly recognized as the main cause for the high failure rate observed. This can be easily perceived looking at the right portion of Figure 19. Use of this type of structural system has been discontinued since 1985.


Figure 19. Typical waffle-slab multi-story building and collapse of a similar system (Mexico City, 1985)

- *Upper-story failure*

Upper-story failures, as that shown in Figure 20, accounted for 38 percent of the cases of severe damage observed in buildings during the 1985 Mexico City earthquake. The ground motion at the sites where seismic damage was highest, the energy content of the ground motion was concentrated within a narrow band centered at a dominant period of 2 seconds, which is longer than that of most of the buildings affected. The ordinates of the pseudo-acceleration response spectra were significantly lower for the higher natural modes than for the fundamental mode. Therefore, damage at the upper stories cannot be attributed to the contribution of higher modes of vibration; the influence of soil-structure interaction had probably a significant role in the observed performance pattern.

At least part of the damage shown in Figure 21 can be attributed to the out-of-plane response of the infill masonry panels to the horizontal accelerations perpendicular to them; these accelerations are highest at the uppermost stories.

Figure 20. Upper story failures (Mexico City, 1985)

Figure 21. Out-of-plane collapse of large and slender brick masonry panels in Gandhidham (Gujarat, 2001; Murty *et al*, 2002)

- *Structural connections and details*

A very large percentage of the cases of significant structural damage or failure is associated with deficient connections and reinforcement details.

Figure 22a, taken from Maison & Hale (2004), shows details of a typical beam-column connection of a steel-structure building that experienced severe damage during the 1994 Northridge earthquake. A very large concentration of welds existed right behind the moment-connection girder-to-column flange welds. Fractured connections were discovered during a building survey conducted shortly after the earthquake. A sketch of a typical failure pattern is depicted in Figure 22 b) cracks were found in either the weld metal or the steel heat-affected zone next to the weld. In some cases, as that shown in the figure, cracks crossed the whole columns flange and web, thus taking the system to a condition very close to failure.

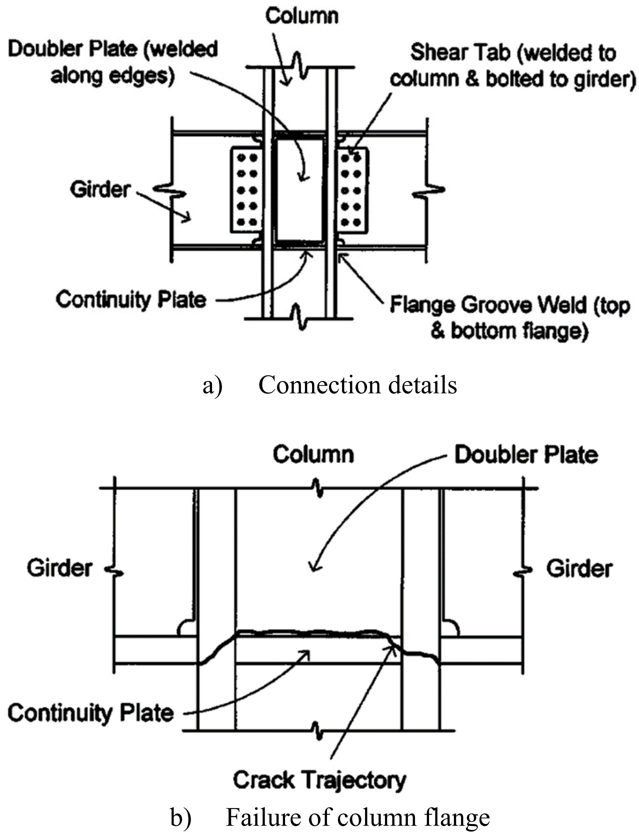


Figure 22. Brittle failure of steel beam-to-column connection
(From Maison *et al*, 2004)

Figure 23, taken from Gur *et al* (2009), shows the failure of a reinforced concrete column under the combined action of axial compression and diagonal tension. This is easily understood, looking at the very low ratio of transverse reinforcement shown in the figure. These types of faulty details and severe structural failure are usually observed during post-earthquake inspections all around the world.

Figure 23. Failure at top of RC column, associated with poor reinforcement details (Bingöl, 2003; from Gur *et al*, 2009)

- *Failure of infill masonry panels*

Unless they are isolated from the main frame, infill masonry panels are often subjected to significant levels of in-plane shear stresses. They interact with the confining frames as compression struts, and they are subjected to significant tension stresses in the direction of the other diagonal. A typical failure of an element of this type is illustrated in Figure 24: the thrust of the compression strut on the corners of the frame providing the confinement produced diagonal tension failure at the bottom of the column at the left and at the top of the column at the right. These local failures contributed to increase the tension stresses in direction of the other diagonal, and to the development of the diagonal crack shown in the figure.

- *Deficient structural arrangement*

The building shown in the left side of Figure 25 did not have the ground-floor column corresponding to the corner appearing at the right end of the figure. The collapse of the whole system was probably associated with a mechanism that started developing when the columns at that end of the building failed under the action of the local overturning moment.

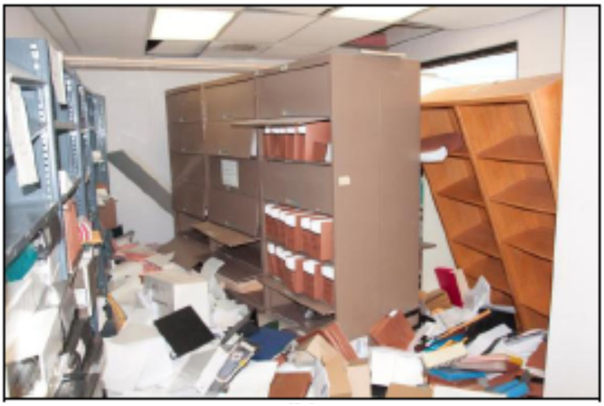
Figure 24. Two combined failure mechanisms for a masonry panel: diagonal tension both in panel and in confining frame
(Adana-Ceyhan, Trukey, 1999; from Bachmann, 2003)

Figure 25. Collapse of tall building with discontinued structural wall at one corner (Concepcion, Chile, 2010; from Elnashai *et al*, 2010)

- *Non-structural damage: contents*

A significant portion of the economic losses produced by earthquakes is associated with non-structural damage. Many of them occur by breaking of window glasses or by cracking of infill wall panels; these are often prevented by means of adequate connecting details between structural and non-structural elements. However, actions oriented to controlling the losses associated with damage on contents and equipment are often neglected, in spite of the fact that they can be at least as important as those arising from damage on the elements of the construction. The measures needed to prevent losses of the types shown in

Figures 26a-b are very easy to implement; also, they lead to a very high benefit/cost ratio.



a) Collapse of shelves (Mexicali, 2011; from Meneses, (2010)

b) Collapse of water tank on top of RC building (Bhul, 2001; from Murty *et al*, 2002)

Figure 26. Non-structural damage due to overturning or sliding of furniture, equipment or objects

3 EARTHQUAKE-RESISTANT DESIGN CRITERIA FOR BUILDINGS: CURRENT CHALLENGES AND TRENDS

3.1 An overview of current challenges and trends

For many years, the objectives of Earthquake Engineering have been clear to the professionals working in the design and construction of structures at sites exposed to significant seismic hazard:

a) Prevent system collapse under the action of high-intensity earthquakes, expected to occur at long return intervals (several centuries),
b) Control structural and non-structural damage that may be produced by moderate-intensity earthquakes (return intervals of several decades), and prevent all types of damage under the action of low-intensity earthquakes, associated with short return intervals (a few decades). In order to comply with these objectives, attention has been focused on requirements about lateral strength and stiffness properties of structural systems.

During the last decades, the availability of powerful computer tools for the nonlinear dynamic response analysis of complex structural systems, as well as the experiences derived from the observations about the seismic performance of constructions, have fostered the development of an approach to earthquake engineering based on the achievement of specified expected performance targets. This approach has replaced the former one, based on the compliance with lateral strength and stiffness requirements. Figures 27 and 28, taken from Deierlein (2004) and Calvi (2010), respectively, show paradigms well recognized by the global engineering community about the basic concepts related to performance indicators and expected seismic performance targets. In Figure 27, IO, LS and CP stand for *immediate occupancy, life safety and collapse prevention*, respectively. According to the Structural Engineers Association of California (SEAOC), the return intervals proposed in Figure 28 (taken from Calvi, 2010) for the target performance levels shown in the horizontal axis are 43, 72, 475 and 975 years for frequent, occasional, rare and very rare events, respectively (SEAOC, 1999). These numbers are in contradiction with a later proposal by the American Society of Civil Engineers (2006b), which places its emphasis on two probabilistic earthquake levels: return intervals of 475 and 2475 years for LS and CP performance targets, respectively.

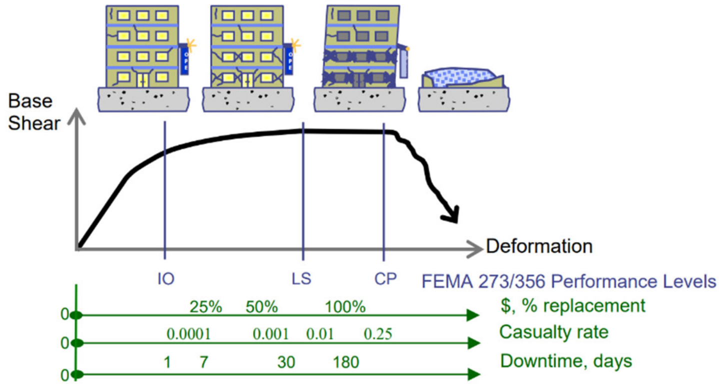


Figure 27. Quantitative indicators of expected performance (Deierlein, 2004)

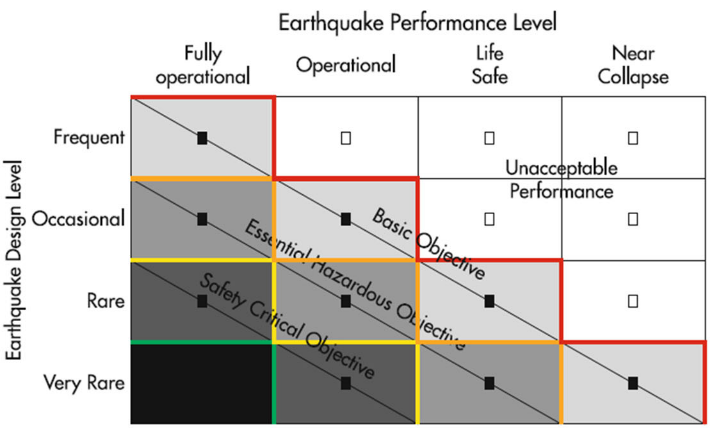


Figure 28. Expected performance *vs* seismic exposure (Calvi, 2010)

An important engineering challenge rises immediately: that of estimating expected performance levels for a given ground motion intensity and/or structural response. As mentioned by Naeim (2010), FEMA-356 (Federal Emergency Management Agency, 2000) and ASCE 41-06 (American Society of Civil Engineers, 2006a) have recognized four possible analytical procedures to estimate dynamic response demands of structural systems to seismic excitations:

a) linear elastic static analysis (LSP), b) linear dynamic procedure (LDP), commonly carried out in terms of response spectrum analysis, c) nonlinear static procedure (NSP), known as push-over analysis, and d) dynamic nonlinear response analysis (DNP). The response estimates resulting from each of these types of analysis are affected by two types of uncertainties: aleatory and epistemic; the former is associated with the random nature of the seismic excitation, and the latter with our imperfect knowledge of the system properties or with the limitations of the models used to predict its behavior under a given excitation. Because reliability-based performance targets are ordinarily expressed in terms of the probabilities of occurrence of structural response demands larger than the system capacity corresponding to an associated limit state, it becomes necessary to count with information about the epistemic uncertainties associated with each of the structural analysis procedures described above.

In practical applications, performance objectives must be established both at the global and at the local levels, describing the expected damage and the failure probabilities for the system as a whole and for each individual member or critical section, respectively. The expected global response and performance of a structural system subjected to a high-intensity earthquake excitation is very sensitive to its deformation and energy-dissipation capacities, which are strongly dependent on the dominant global failure modes and mechanisms. This has motivated the development of "*capacity design*" criteria. According to them, both the basic structural arrangement and the local safety factors for the different members and critical sections are determined so as to ensure the dominance of ductile global failure modes; *i.e.,* associated with the possibility of experiencing several cycles of large lateral displacements, without collapse, after the maximum base shear-force capacity is reached.

The following sections present a brief discussion of the concepts introduced in the foregoing paragraphs. Detailed descriptions of the current trends along different lines in Earthquake Engineering can be found in a book edited by Bozorgnia and Bertero (2004).

3.2 Seismic hazard assessment

According to Esteva (1976), the seismic hazard at a site is given by the following equation, which accounts for the contributions of potential seismic sources at short distances from the site:

$$\nu_Y(y) = \iint \left|\frac{d\lambda_M(m,x)}{dm}\right| P(Y \geq y|m,x)dmdV \qquad (1)$$

Here, $\lambda_M(m,x)$ is the rate of occurrence, per unit of time and unit of volume, of earthquakes with magnitude equal to or larger than m; its value depends on the vector x of coordinates of the elementary volume dV included in each of the seismic sources that contribute significantly to $\nu_Y(y)$. $P(Y \geq y|m,x)$ is the probability that an earthquake with magnitude m, generated at a point with coordinates x, leads to an intensity equal to or larger than y at the site of interest. Its value can be easily estimated by means of the intensity attenuation functions, valid for the zone and the site of interest. Those functions must take into account the influence of the local conditions, such as pronounced topographical irregularities or the presence of soft soil layers. The double integral must cover all possible values of the magnitude and all the volume of the earth's crust, near the site, which may contribute significantly to $\nu_Y(y)$, the seismic hazard at the site.

3.2.1 Activity of potential seismic sources near a site of interest: probabilistic models

Available samples of statistical information about the occurrence of moderate- and large-magnitude earthquakes within the area of significant influence for the estimation of probabilistic seismic hazard functions at a given site are usually too small to provide accurate estimates of those functions. The need to account for the large epistemic uncertainties associated with the estimation of local seismicity functions, or rates of occurrence of earthquakes with magnitudes greater than different given values, was recognized by Esteva (1969), who presented a Bayesian framework for the probabilistic estimation of those functions. The criterion is based on the adoption of a priori subjective probabilistic assumptions about the variability of the local seismicity in a small portion of a given potential seismic source with respect to its average value in a larger system with similar geo-tectonic characteristics. Bayes theorem is then applied, in order to modify the a priori assumptions by introducing the available statistical information in the small region that is relevant for the estimation of seismic hazard at the site of interest. This is a reasonable manner of making use of both, statistical and physical information, for the development of uncertainly defined probabilistic models of the activity of potential seismic sources in the neighborhood of a site of interest. Criteria to account for this uncertainty in the formulation of optimum reliability-based engineering decisions were proposed by Rosenblueth (1976); Jalayer and Cornell (2003) derived approximate expressions to include it in a framework for probability-based demand-and-capacity-factor seismic design formats.

Uncertainties associated with the possibility of generation of earthquakes of significant magnitudes at previously unidentified seismic sources are much more difficult to handle. The possibility of existence, or birth, of those sources of hazard and risk must be carefully considered, at least by subjective comparison with the experience derived from observations at sites located within similar geo-tectonic environments.

A summary has been presented in Section 2.1 about the information provided by Naeim (1995), related to the wide dispersion and the extremely high values of the ground motion intensities recorded at sites close to the source of the Northridge 1994 earthquake. Mention is also made of the significant differences in the duration and the frequency content of the ground motion that may result from the directivity effects. The ratios between the ordinates of the linear and the elasto-plastic pseudo-acceleration response spectra shown in Figure 4 differ significantly from those generally used to propose spectral reduction factors for practical structural design applications. A careful examination of the figure shows that the differences may be both on the conservative and on the non-conservative sides. Several significant engineering challenges arise from the considerations presented in the foregoing paragraph. These challenges include the following ones, among others:

a) Deriving intensity-attenuation functions applicable to sites located within the seismic source or very close to it,
b) Obtaining sufficient samples of actual or artificial ground motion time histories with duration and frequency-content properties representative of those likely to occur in the immediate vicinity of a potential seismic source,
c) Using these time histories to revise current criteria for the derivation of reduction factors of linear response spectra, applicable to the development of design-oriented response spectra that account for nonlinear response and hysteretic energy dissipation.

3.2.2 Ground motion models and intensity measures

Most of the seismic reliability studies proposed for the implementation of performance-based earthquake-resistant design criteria assume that the seismic excitation consists of one single horizontal ground motion component acting on a two-dimensional structural system or on the vertical plane of symmetry of a symmetric system. Therefore, generally available models and algorithms for the generation of artificial ground-motion time histories are formulated to deal with this type of excitation. However, in actual cases typical of engineering practice, for high intensity values the dynamic response of the system is nonlinear and the strength and stiffness properties of some members or critical sections for lateral forces in the direction parallel to one of the components is sensitive to the internal forces associated with lateral forces acting in the orthogonal direction. For these cases, performance-related studies must be based on the dynamic response of the system under the action of samples of pairs of both simultaneous orthogonal components. If I_X and I_Y denote respectively the ground motion intensities (measured by an adequate indicator) of the components in the X and Y directions, the intensity of the seismic event, including both components, can be represented, for instance, by the quadratic mean, $I = \sqrt{(I_X^2 + I_Y^2)/2}$. The ratios I_X/I and I_Y/I are then handled as random variables with probability density functions estimated from the values contained in a sample of pairs of actual observed intensities at the site of interest. This approach has been applied by the author in a study now in process about the seismic reliability functions for in-plan asymmetric multistory buildings. Similar criteria should be available in the near future for the simultaneous consideration of the vertical component and the two horizontal ones.

The seismic response of multi-support structures, such as bridges, and of infrastructure networks, such as hydraulic pipe systems, is strongly related to the spatial variation of the ground motion intensity, which must be represented by the probabilistic correlation of those intensities, not only by their local values. The spatial variation of the local intensities is strongly dependent on the spatial variation of the soil mechanical properties and the topographic configuration. Samples of the statistical information needed to derive spatial variation functions are usually not available; facing this problem requires the development of adequate physical models, which in turn requires the availability of extensive information about the mechanical properties of the ground at a wide space around the foundation of the system of interest.

3.2.3 Intensity attenuation equations (including near the source)

The first probabilistic seismic hazard maps on firm ground in Mexico were published by the author in 1969, with the final aim of using them for the development of seismic design regulations for seismic excitations with intensities corresponding to given return intervals. Two complementary measures of the ground motion intensity were adopted for this purpose: peak values of the ground velocity and of the ground acceleration in a given direction; the corresponding pseudo-acceleration response spectra for a damping coefficient of 0.05 were estimated in terms of these two parameters according to an expression proposed by Newmark and Rosenblueth (1971). For a given site and a given seismic event, this information is generally not available; this generated the need for the development of intensity attenuation functions in terms of information about the magnitude of the earthquake and the source-to-site distance. Equations 2 and 3 were proposed by Esteva and Rosenblueth (1964), using information available from acceleration records of earthquakes generated by several seismic sources in California, USA.

$$v = 15e^{M}(R + 0.17e^{0.59})^{-1.7} \tag{2}$$

$$a = 1230e^{0.8M}(R + 25)^{-2} \tag{3}$$

In these equations, a (cm/s^2) and v (cm/s) are the peak absolute values of ground acceleration and velocity, respectively, M is the magnitude of the earthquake and R (km) is the source-to-site distance.

Alternate measures of seismic intensity have been proposed for practical applications. For instance, Alamilla and Esteva (2006) proposed the use of the ordinate of the pseudo-acceleration linear response spectrum for the fundamental natural period of the system or for that corresponding to a simplified reference system. The authors used this measure of intensity for the development of seismic reliability functions for multistory buildings, taking into account uncertainties associated with the characteristics of the seismic excitations, as well as those corresponding to the mechanical properties of the structural members and to the permanent loads. Uncertainties associated with the characteristics of the seismic excitations include, among others, those related to the acceleration time histories for a given intensity, corresponding to a specified return interval. Dealing simultaneously with all uncertainties required the application of a Monte Carlo simulation process for both, the ground-motion time histories and the properties of the structural system,

3.2.4 Influence of local conditions: soil mechanical properties, topographic configuration

Intensity attenuation functions constitute very valuable tools for the assessment of seismic hazard at sites where direct instrumental information is not available, or not large enough, about the ground-motion time histories during high intensity seismic events. However, their applicability to a specific site is valid only if the local conditions are similar to those of the sites where the instrumental information used for their determination was obtained. For this reason, seismic hazard functions at sites not complying with this requirement can be determined as follows:

a) Determine the hazard function at the site of interest, assuming firm ground and flat topographic configuration,
b) Include the influence of local conditions, by means of the following equation:

$$\nu_Y(y) = \int \left|\frac{d\lambda_U(u)}{du}\right| P(Y \geq y|u)du \tag{4}$$

Here, *Y* is the intensity at the site considered, including the influence of local conditions, and *U* is the intensity at the same site, neglecting the influence of local conditions. The function $P(Y \geq y|u)$ can be estimated by means of a simulation procedure of the responses of a mathematical model of the local properties of the site, when subjected to a sample of seismic excitations arriving from the seismic source. (Alamilla *et al*, 2001a).

3.2.5 Monte Carlo simulation of samples of ground-motion time histories

For the simulation of samples of ground-motion time histories, for a given intensity at a given site, it is necessary to count with models about the evolutionary properties of those time histories. These models must take into account the magnitude and the site-to-source distance of each seismic event, as well as the local ground properties at the site, such as type of soil and topographic configuration. A mathematical model to deal with this problem was proposed by Alamilla *et al* (2001a), who presented an expression to represent those properties. They applied it to the generation of a sample of ground motion records at a soft soil site Mexico City, with a specified value of the intensity, the latter measured by the ordinate of the pseudo-acceleration response spectrum for a given natural period, usually that corresponding to the fundamental period of the system of interest. Ismael and Esteva (2006) used this information to develop a hybrid method of simulation of ground motion records, with information derived both from actual records and from others generated as the summation of the contributions of record of low intensity earthquakes, used as Green's functions.

In many practical applications, it is necessary to simulate pairs of simultaneous horizontal orthogonal components of seismic ground motion records. For this purpose, the seismic intensity of each event is measured by $S_a(T)$, the quadratic mean of the ordinates of the pseudo-acceleration response spectrum for each of the two simultaneous components. This process takes into account the probabilistic correlation between the ordinates corresponding to both components, S_{aEW}^2 and S_{aNS}^2 (Alamilla *et al*, 2001b).

$$S_a(T) = \sqrt{\frac{S_{aEW}^2 + S_{aNS}^2}{2}} \tag{5}$$

Figure 29 presents an example of the results of the application of this approach to the Monte Carlo simulation of pairs of orthogonal horizontal components of ground motion records at a soft soil site in Mexico City, for an intensity corresponding to a return interval of 125 years, for a structure with a natural period T = 0.8s. The probabilistic correlation between both components was taken into account with the aid of samples of the auxiliary variable R_{Sa}, defined as follows:

$$R_{Sa} = \frac{S_{aEW}}{\sqrt{\frac{S_{aEW}^2 + S_{aNS}^2}{2}}} \tag{6}$$

Figure 30 shows the expected values and the standard deviation of R_{Sa} obtained for different values of the fundamental period of vibration of the structural system considered. This information was used for the Monte Carlo simulation of samples of pairs of simultaneous horizontal orthogonal components for earthquakes with intensities corresponding to previously selected return intervals. Figure 31 shows an example of the two horizontal ground motion records and the pseudo-acceleration and displacement response spectra, for a damping ratio of 0.05, for an intensity corresponding to a return interval of 50 years.

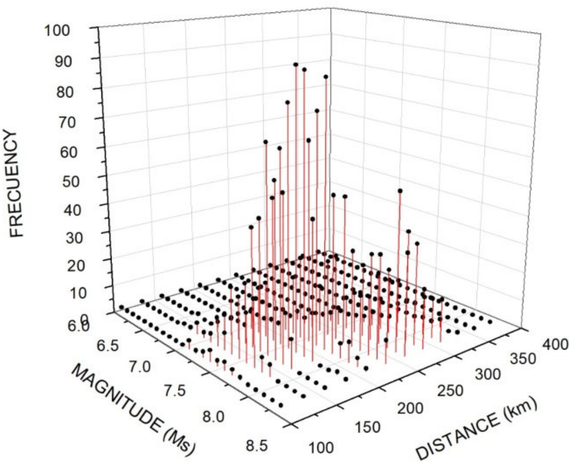


Figure 29. Joint probability density function of magnitude and distance for a given intensity at a site

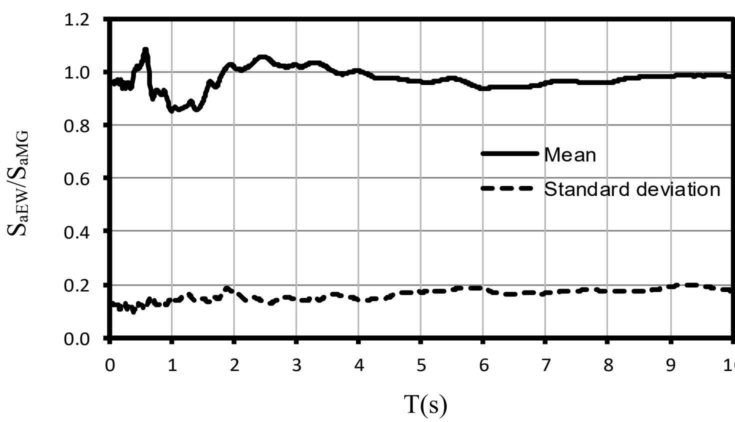


Figure 30. Relation of intensity of the EW component to the quadratic mean, for a sample of earthquake records

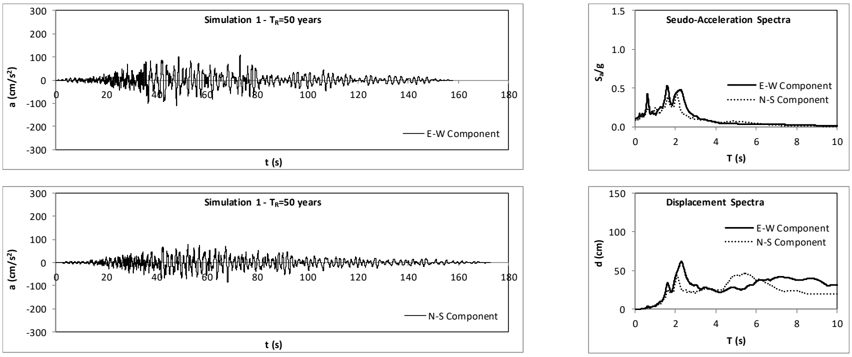


Figure 31. Simulated acceleration record for an intensity corresponding to a return interval of fifty years

3.3 Seismic vulnerability analysis

In general, the seismic vulnerability function of a system can be expressed as follows:

$$\bar{\delta}(y) = E[\delta|y] = p_F(y)\delta_F + \left(1 - p_F(y)\right)\bar{\delta}(y|S) \qquad (7)$$

In this equation, y is the intensity of the ground motion affecting the system, $p_F(y)$ is the probability of collapse, conditional to the value of the intensity, δ_F is the cost of the consequences of collapse and $\bar{\delta}(y|S)$is the expected value of the cost of damage, in case of survival.

In all cases, the expected-cost values presented are normalized values, obtained dividing the actual nominal costs by the initial construction cost, C_0. Function $\delta(y \mid S)$ depends on the probability distribution of the physical damage experienced by all structural and non-structural components of the system, as well as on the direct and indirect consequences of that damage. Its evaluation must also account for potential damages experienced by equipment, installations or contents that may fail, for instance, by overturning or by excessively high dynamic response. Some illustrative examples about the application of Equation 1 to life-cycle optimization studies of buildings located at sites with significant seismic hazard conditions are presented by Esteva *et al* (2002, 2011).

In the following, attention will be focused on the determination of the seismic reliability function ($1-p_F(y)$), which is equal to the probability that the seismic capacity of the system (for instance, lateral distortion capacity) is greater than the value of the corresponding seismic response demand (peak absolute value of lateral distortion). Alternatively, the seismic capacity can be measured by the value y_F of the intensity that produces failure of the system and the seismic response demand by the value of the acting intensity, Iy (Esteva *et al*, 2010a). Other possible definitions of these variables include concepts of low cycle fatigue, energy dissipation and damage accumulation.

3.3.1 Seismic vulnerability indicators

In many practical applications, it is convenient to work with the reliability index $\beta(y) = \bar{Z}(y)/\sigma_Z(y)$ (Cornell, 1969), instead of determining directly the reliability function, $\left(1 - p_F(y)\right)$. Here, $\bar{Z}(y)$ and $\sigma_Z(y)$ are respectively the mean value and the standard deviation of the safety margin Z with respect to failure of the system subjected to an earthquake of intensity y and uncertainly known detailed time history. For this purpose, Z is defined as the natural logarithm of the ratio of the seismic capacity of the system to the corresponding seismic response demand. Given $\beta(y)$, $p_F(y)$ can be estimated by means of the approximate relation $p_F(y) = \Phi\left(-\beta(y)\right)$, where $\Phi(\cdot)$ is the Normal Standard probability distribution function.

The following paragraphs summarize the results of studies about the sensitivity of the reliability function $\beta(y)$ to different mechanical properties and global parameters of the structural system.

- *Capacity design (selection of failure mechanisms)*

The importance of avoiding the occurrence of brittle failure modes is one of the main lessons learned from the observed seismic performance of actual structural systems. Capacity design criteria respond to that lesson; their application includes essentially the following concepts:

a) Select *a priori* the members or critical sections (called "*pre-selected yielding elements*" in the following) that should yield during the development of a failure mechanism,
b) Design them in such a manner as to ensure that they are capable of developing sufficiently high ductile behaviour, and making all other members stronger than required to satisfy equilibrium conditions at their connections with those pre-selected yielding elements

A selection of cases normally considered as pre-selected yielding elements is presented in Table 1, taken from Naeim (2010). Once their location has been selected for a specific structural system, the yield values that characterize the constitutive functions of the corresponding structural members are made equal to those obtained by applying an adequate safety factor to the internal forces resulting from a linear response analysis for the design earthquake. The strengths of the other members are then determined, applying to them a higher safety factor than that adopted for the pre-selected yielding elements. Even when this is done, it may happen that the ductile-failure condition does not dominate, either because of the uncertainties associated with the actual strengths of the different members or critical sections, or because the deformation patterns associated with the dynamic structural response generate local failures at members or sections different from those previously selected. Keeping sufficiently low the probability of occurrence of these conditions must be maintained as an objective of performance-based design criteria.

Table 1. Zones and actions normally adopted to define ductile elements (From Naeim, 2010)

Structural system	Zones and actions
Special moment resisting frames (steel, moment or composite)	Flexural yielding of beam ends (except for transfer girders) Shear in beam-column panel zones
Special concentric-braced frames	Braces (yielding in tension and buckling in compression)
Eccentric braced frames	Shear link portion of the beams (shear yielding preferred, but combined shear and flexural yielding permitted)
Un-bonded braced frames	Un-bonded braced cores (yielding in tension and compression)
Special steel-plate shear walls	Shear yielding of web plates Flexural yielding of beam ends
R/C shear walls	P-M yielding at the base of the walls (top of foundation or basement podiums) or other clearly defined locations with plastic hinge region permitted to extend to a reasonable height above the lowest plane of a nonlinear action as necessary Flexural yielding and/or shear yielding of link beams

It must be recognized that ensuring the dominance of ductile failure modes may not be feasible for some types of structural arrangements, which provide the optimum solution for some specific constructions. Take, for instance, a low rise building whose lateral strength is supplied by masonry walls confined by pilasters and bond beams. Diagonal-tension cracking will always be the critical failure mode; brittle behaviour can be mitigated, but not completely avoided, by adding steel reinforcement at the horizontal joints between successive brick layers.

Moderate levels of ductile behaviour may be achieved, provided the confining members are strong enough to support the forces transmitted by the masonry walls, acting as compression diagonals. The resulting low values of the lateral deformation capacities must be compensated by the adoption of sufficiently high base-shear strength requirements.

- *Global and local performance indicators in multi-story buildings*

The overall drift ratio, defined as the quotient of the roof displacement relative to the base, divided by the building height, is usually taken as an indicator of global system performance. It must be compared with its acceptable value, determined by means of a conventional push-over analysis or by a generally specified value.

For instance, PEER (2010) Guidelines limit overall drift ratio to 0.005 for serviceability (IO) conditions and 0.03 for the collapse prevention (CP) condition. In addition, inter-story drift must be limited to 0.045 for the latter condition. Clearly, the specified values are not applicable to dual wall-frame or to slender systems (high height-to-width ratio). For the former cases, the lateral deformation capacity is dominated by that of the shear wall segment at each story; for the latter cases, inter-story drifts include two components: the story distortion and a story rotation with respect to a horizontal axis. In these cases, the global deformation capacity must be determined by means of a pushover analysis.

Local ductility demands and deformation capacities of bracing members can easily be estimated by ordinarily applied methods of structural analysis; however, this is not true for other types of ductility demands, such as the post-yielding bending curvatures at the ends of beams or columns, which can only be determined by means of a step-by-step nonlinear dynamic response analysis. Because in many practical cases this method of analysis is not applied, performance-acceptance conditions specified for response demands estimated by other methods must lead to sufficiently low values of the probability of exceedance of local deformation capacities at individual members or critical sections.

Efforts to control non-structural damage have been focused on some concepts, such as cracking of partition walls or infill elements and breaking of glass panels, which are sensitive to story distortions. However, they have practically ignored the possibility of controlling the damage associated with overturning of furniture, shelves and objects, which are sensitive to the local floor accelerations or velocities, in spite of the fact that the criteria needed to estimate the occurrence of this type of damage has been available for a long time (Ishiyama, 1981).

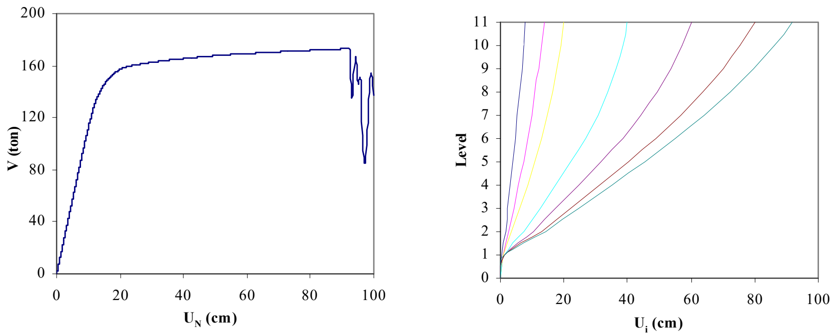
- *Influence of soil-structure interaction*

At soft-soil sites, the lateral displacements of a building result from the superposition of the lateral distortions of the structural system and from the global rotation associated with the rotation of the base, due to the flexibility of the soft-soil foundation.

- *Reliability-based design*

As mentioned at the start of this Section (3.3), attention here will focus on the criteria to estimate the seismic reliability of a system for a given earthquake intensity, with uncertainly known detailed characteristics of the ground motion time history. The next few paragraphs are based on Esteva *et al* (2010).

Current approaches for the determination of the seismic reliability of a system subjected to an earthquake ground motion with a specified intensity propose to measure that reliability by the probability that Ψ, the peak absolute value of the global distortion, is smaller than the deformation capacity, Ψ_C. Approximate estimates of second-moment probabilistic indicators of Ψ_C are often obtained with the aid of a simplified reference system, characterized by mechanical properties determined by means of a static pushover analysis of the detailed system (See Figure 32).



a) Base shear *vs* roof displacement for detailed model

b) Lateral response configurations of detailed model

Figure 32. Results of pushover analysis for a ten-story building used for illustration

However, these estimates may include significant uncertainties, because according to this approach it is not possible to account for a) the influence of cumulative damage associated with the cyclic response, and b) the dependence of the lateral deformation capacity on the response configuration of the system when it approaches failure. Figure 32 b) shows an example of the possible variation of this configuration, according to the amplitude of the top displacement.

Trying to avoid the introduction of arbitrary assumptions about the determination of the deformation capacity of a complex system, in order to obtain reasonable estimates of its seismic reliability, alternative criteria are available, according to which system failure is assumed to take place when the displacements predicted by the dynamic response analysis become indefinitely large and nonreversible. The effective values of the elements of the resulting stiffness matrix are then infinitely small. This condition is described as system collapse (Esteva, 1992; Shome and Cornell, 1999; Alamilla and Esteva, 2006). In order to determine the safety factor of a given system with respect to this type of failure for a given ground motion time history, it is necessary to obtain the scale factor that has to be applied to that time history in order to produce system collapse. The intensity leading to collapse is then denoted as *"failure intensity"*. Because the determination of the needed scaling factor requires the use of an iterative procedure, it may call for excessive demands of computer time.

The method of Incremental Dynamic Analysis (IDA, Vamvatsikos and Cornell, 2002) offers both possibilities for the estimation of probabilistic indicators of seismic reliability for given ground motion intensities: either on the basis of deformation capacities or using the concept of failure intensity. However, these advantages are often tied to excessive computer time demands (Dolsek and Fajfar, 2004). This has led Esteva and Ismael (2004) and Esteva and Díaz-López (2006) to explore an alternative approach, aiming at estimating reliability functions relating the reliability index β (Cornell, 1969) with the ground motion intensity, including the influence of cumulative damage and avoiding the need to obtain probabilistic definitions of lateral deformation capacities. This approach is based on the concept of *"failure intensity"*, mentioned above. According to this approach, the collapse condition is expressed in terms of a secant-stiffness reduction index $D_k = (K_0 - K)/K_0$, where K_0 is the initial tangent stiffness associated with the base-shear *vs* roof displacement curve resulting from pushover analysis. K is the secant stiffness (base shear divided by lateral roof displacement), when the lateral roof displacement reaches its maximum absolute value during the seismic response of the system. The failure condition is expressed as $D_k = 1.0$.

According to the approach proposed by Esteva and Díaz-López (2006) and Esteva *et al* (2008), a reliability function $\beta(y)$ is obtained from a sample of pairs of values of D_k and y, where β is the safety index proposed by Cornell (1969) and y is the ground motion intensity. If the sample includes only cases with D_k smaller 1.0, the reliability function can be obtained by means of a regression analysis; if cases with $D_k = 1.0$ are also included, a maximum likelihood analysis must be performed. Instead of formulating the problem as that of obtaining an indicator of the probability that $D_k < 1.0$ (survival) for a given intensity, attention is focused on the determination of second moment indicators of the probability distribution of $Z_F = \ln Y_F$, where Y_F is the minimum value of the intensity leading to the condition $D_k = 1.0$ (collapse). For an earthquake with intensity equal to y, a safety margin Z_M can be defined equal to the natural logarithm of the ratio of the system capacity to the amplitude of its response to the given intensity; it can also be defined as the natural logarithm of the ratio Y_F/y. The reliability function can then be expressed in terms of the index $\beta(y)$ presented above.

For the structural system of interest, a sample of pairs of random values of Z and the stiffness reduction index, D_k, can be used to estimate means and standard deviations of $Z(u)$, the latter defined as the natural logarithm of the random intensity Y that corresponds to $D_k = u$. According to the ranges of values of D_k included in the sample, the estimation can be performed either by means of a conventional minimum-squares regression process or through a maximum likelihood analysis, as proposed by Esteva *et al* (2010).

Figure 33 (taken from Rangel, 2006), was presented by Esteva and Díaz (2006). It shows a plot of the values of the normalized earthquake intensities $S_a M/\bar{V}_y$, leading to different values of the stiffness reduction index, $D_k = (K_0 - K)/K_0$, for a twenty-story system with hysteretic energy dissipating devices (denoted by EDD's in the following), subjected to a set of synthetic ground motion records at a soft soil site in the Valley of Mexico. Here, S_a is the linear pseudo-acceleration response ordinate for the fundamental period of the system of interest, M the mass of that system and $\bar{V}_y$ the yield value of the base shear force determined through a pushover analysis using the expected values of the mechanical properties of the structural members that constitute the system. The records were simulated with the aid of the hybrid algorithm presented by Ismael and Esteva (2006). As shown in the figure, the results for values of D_k smaller than 1.0 were used to estimate the mean and the standard deviation of the natural logarithm of the failure intensity. The resulting reliability functions for three different combinations of the relative contributions of the reinforced concrete frame and the energy dissipating devices are shown in Figure 34. The values of $\beta(y)$ obtained following this approach were slightly higher than those obtained using the method of incremental dynamic analysis proposed by Vamvatsikos and Cornell (2002).

In spite of the limitations of the criteria and methods used to define a lateral deformation capacity for a structural system, it is very likely that the approach of assessing the seismic reliability of nonlinear systems by comparing lateral displacement demands with the corresponding deformation capacities will offer significant conceptual advantages for the practice of earthquake resistant design. Currently, acceptable values of lateral deformation capacities recommended in normative documents are estimates based on engineering judgment. This fact points at the convenience of developing studies oriented at deriving seismic failure probability functions in terms of probabilistic estimates of dynamic response demands of different types of structural systems and arrangements.

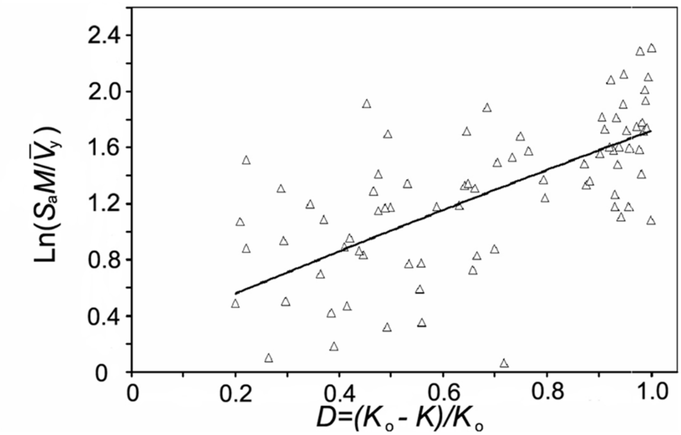


Figure 33. Reliability function in terms of normalized intensity for several twenty-story frame buildings

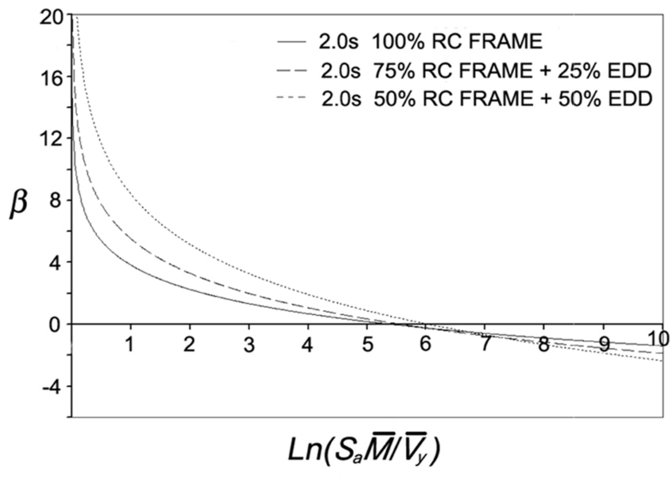


Figure 34. Normalized intensity *vs* β for 20-story system with energy-dissipating devices (EDD's) with EDD's

3.3.2 Influence of structural system configuration

Within the context of building structural systems, three main types of irregularities are often identified in the process of establishing codified recommendations for earthquake resistant design:

Type I: Vertically irregular systems, characterized by significant variations of the floor masses, the story strength and stiffness properties and the safety factors with respect to story shear associated with linear response.

Type II: In-plan irregular systems, characterized by significant eccentricities of the story shear-forces determined by linear dynamic response analysis with respect to the corresponding centres of lateral strength and stiffness.

Type III: Slender systems, characterized by height/with ratios greater than 2.5.

Some brief comments about the influence of irregularities of type I on the seismic response demands and on the seismic reliability-functions of buildings are presented in the following. Types II and III systems, and those characterized by more than one type of irregularity, are very frequent, but up to now they have not been the object of systematic studies oriented to estimating their reliability and performance functions.

Many other types of irregularities can be identified, which may have a significant influence on the dynamic response demands and reliability functions of tall buildings. To name just a few: large in-plan openings, recessed and salient areas, large length/width ratio, non-rigid floor system, sharp along-height variation of floor area or weight, lack of lateral restriction to columns at floors with recessed areas, etc.

- *Vertically irregular systems*

Among vertically irregular systems, buildings with free ground story have received the greatest attention in seismic response and reliability studies, motivated by the empirical evidence about their high seismic vulnerability levels, mentioned in Section 2. The following paragraphs summarize the results of a study related to this topic (Díaz-Alcántara, 2008).

Two sets of symmetric-in-plan tall buildings, similar to that described in Figure 35, located at the soft soil site SCT in Mexico City, have been considered in the first stage of a parametric study about the seismic reliability functions of buildings with free ground story. Figure 36 shows the average ordinates (in the NS and EW directions) of the 5% damping pseudo-acceleration response spectra at several sites in Mexico City, for the earthquake of 19 September 1985; the intensities shown in the figure correspond to a return interval of 125 years. For the SCT site, the frequency content of the ground motion is concentrated in a segment characterized by a dominant period of 2.0 sec.

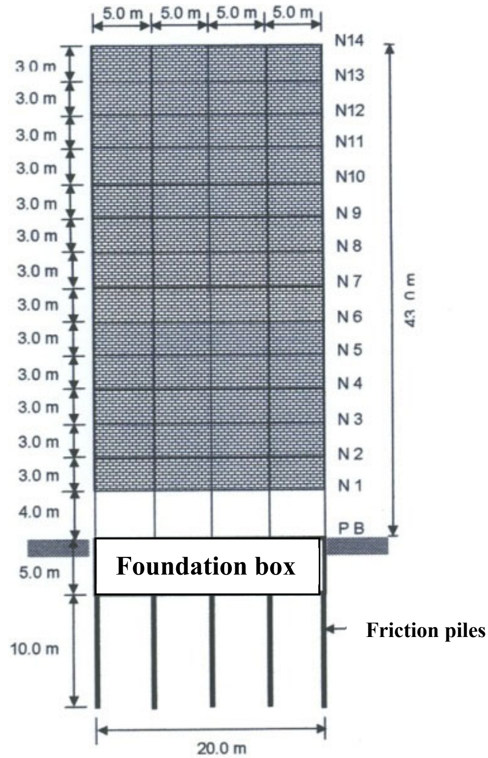


Figure 35. Tall building with free ground story

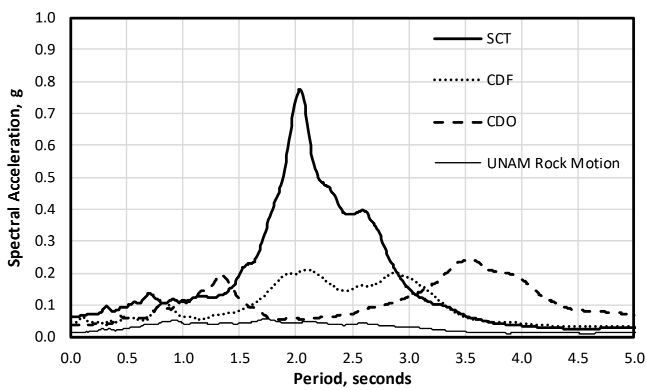


Figure 36. Average pseudo-acceleration response spectra. Different sites in Mexico City, 1985.

Two groups of systems were studied, seven-story and fourteen-story high, respectively. Each group includes an original system, free of any infilling wall panels, designed in accordance with the 2004 edition of Mexico City building code. The properties of other systems in each group resulted from the assumption that all stories included infilling wall panels, with the exception of the ground story. Each group includes several cases, corresponding to different values of the lateral strength and stiffness of each wall panel added to the corresponding original system. Figure 37 shows plots of the seismic reliability functions for the different systems considered, expressed in terms of Cornell's reliability index β, as a function of the normalized intensity, $Z = S_a \cdot \bar{M}_T / \bar{V}_y$. In the figure, S_a is the ordinate of the pseudo-acceleration response spectrum for the fundamental period of the system of interest, $\bar{M}_T$ is the total mass of the system and $\bar{V}_y$ is the yield value of the base-shear *vs* roof-displacement curve determined from the results of a pushover analysis. For the seven-story group of systems, the introduction of the infilling wall panels causes a severe drop in the reliability functions; however, the magnitude of the drop is not sensitive to the values of the strength and stiffness added to the original system. For the fourteen-story systems, the reductions in the value of β are lower, but the influence of the values of the additional strength and stiffness is easily noticed. As expected, the reductions in the values of β grow with the normalized intensity *Z;* that is, with the level of expected nonlinear response.

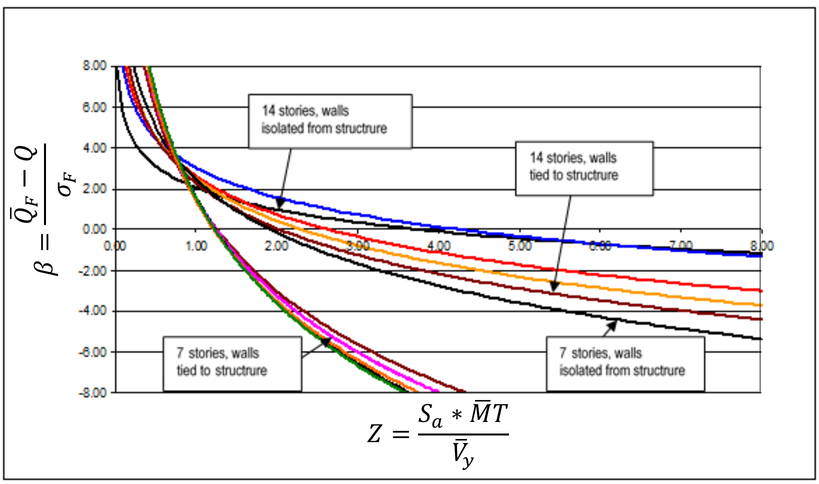


Figure 37. Seven- and fourteen-story frame buildings with walls tied-to and isolated-from the structure: different wall-stiffness values

Additional studies are available about the seismic reliability functions of tall buildings with non-uniform nominal values of the safety factors for story shears along the building height. These studies also show a significant influence of the variability of those safety factors on the seismic reliability levels for the systems considered (López-López, 2008).ditional studies are available about the seismic reliability functions of tall buildings with non-uniform nominal values of the safety factors for story shears along the building height. These studies also show a significant influence of the variability of those safety factors on the seismic reliability levels for the systems considered (López-López, 2008).

- *In-plan irregular systems*

Codified seismic design criteria for in-plan irregular systems are specified in terms of linear dynamic response analysis that account for the joint action of two orthogonal horizontal components. Additional requirements are included to account for accidental torsional eccentricities, which may arise from random variations in the spatial distribution of gravitational loads (mainly live loads) and in the lateral stiffness of the frames or walls that provide the required lateral strength. However, they do not account for the possible amplifications of the ductility demands at the elements at the edges of the system. This may occur for high earthquake intensities, because they are capable of generating high levels of nonlinear behaviour. For a structural arrangement as that shown in Figure 8, the presence of the walls at the building edges opposite to the street facades can generate large strength eccentricities, which can be significantly larger than the corresponding stiffness eccentricities, usually considered in conventional seismic design criteria. No systematic studies are known to the author about the influence of uncertain strength eccentricities on the seismic reliability functions of buildings.

- *Slender systems*

The lateral capacity of typical structural arrangements for slender buildings is ordinarily provided by shear walls or bracing elements. Their global configuration when responding to a seismic excitation is not determined by the lateral story distortions, as in usual, non-slender buildings. It is largely determined, instead, by the global bending response of the building, acting as a large cantilever embedded at the foundation. The ratios of axial compression forces to bending moments in the columns are much larger than those that are typical of non-slender buildings. In order to ensure a high probability of achieving ductile behaviour of the system, the safety factors applied to the shear forces acting on the shear walls, to the axial forces acting on the braces and on the columns must be sufficiently higher than those applied to the bending moments at the column and beam ends. The values of the ratios between the safety factors to be applied to the different types of internal forces must be determined from a reliability-based optimization analysis. This is an important challenge for the development of efficient capacity design criteria.

- *Systems with asymmetric shear-distortion functions*

Some systems, such as those shown in the upper portion of Figure 38, are characterized by asymmetric shear-distortion functions, with different strengths in two opposite loading directions. When these systems are subjected to high intensity ground motion excitations, they tend to accumulate displacements in the weak direction, which leads to peak values of lateral displacements much larger than those experienced by a symmetric system with a force displacement function equal to that corresponding to the weak direction of the asymmetric system. This is evident in Figure 39 (Terán-Gilmore *et al*, 2000), which shows hysteretic response graphs for a cyclic acceleration acting at the base of a tilting system similar to that shown at the upper-right corner of Figure 38. The graphs on the left and on the right sides of Figure 39 correspond to tilting slopes of 0 and 0.005, respectively. Three values of the lateral strength are considered for the system studied, neglecting the influence of tilting: c = 0.20, 0.10 and 0.05; the differences between the peak response amplitudes in both directions grow as the lateral strength decreases, thus contributing to increase the influence of nonlinear behaviour. As it happens in other cases of cyclic responses of nonlinear hysteretic systems, adding strength at one location or member may lead to higher response amplitudes at other elements.

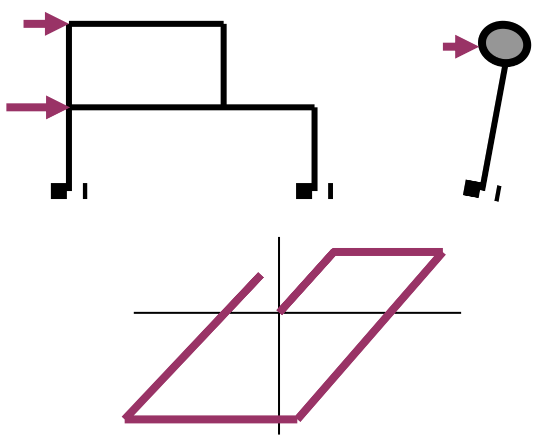


Figure 38. Systems with asymmetric shear-distortion functions

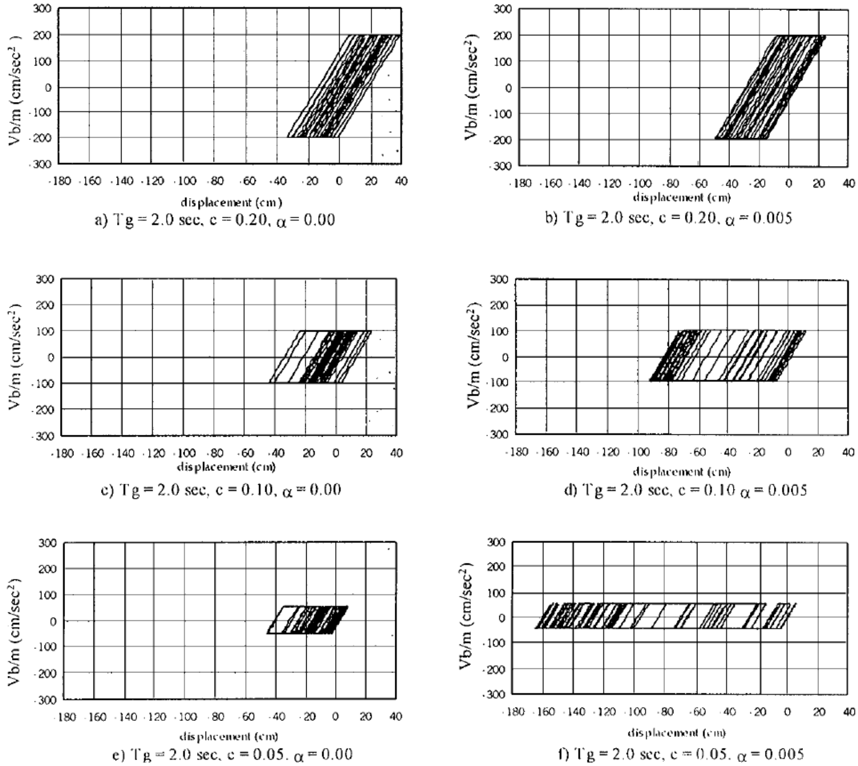


Figure 39. Cyclic response histories of systems with asymmetric shear-***distortion*** functions (from Terán-Gilmore *et al*, 2000)

3.3.3 Soil-structure interaction

Conventional criteria for the estimation of the influence of soil-structure interaction on the seismic response of buildings or other structural systems, non-extended in plan, express this interaction in terms of two types of effects: kinematic and inertial. They both depend on the flexibility of the soil supporting the foundation. The former is a quantitative indicator of that flexibility, while the latter shows the difference between the inertial forces resulting from the structural responses for the conditions of flexible and non-flexible foundation corresponding to soft soil and firm ground, respectively.

Bárcena and Esteva (2007) present the results of a study about the influence of dynamic soil-structure interaction on seismic response, ductility demands and reliability levels for a set of reinforced concrete structures with gravitational loads and mechanical properties (strength and stiffness) representative of systems designed for earthquake resistance in accordance with current criteria and methods.

The buildings are located at soft soil sites in the Valley of Mexico and subjected to ground motion time histories simulated in accordance with characteristic parameters (expected value and dispersion) of the maximum probable earthquake intensity likely to occur during the system's expected life. This intensity was obtained as the mean value of the intensities corresponding to a sample of earthquakes with magnitude M_w = 8.2, generated at a seismic source in the subduction zone, at a distance of 300km. For the near-resonance condition, the effects of soil-structure interaction on the ductility demands depend mainly on radiation damping. According to the geometry of the structures studied, this damping shows a strong correlation with the aspect ratio, defined as the ratio of the building height to its width. For structures with aspect ratio greater than 1.4, the story and global ductility demands are higher than to those obtained for the same structures if built on a rigid base, while for structures with aspect ratio less than 1.4 the ductility demands are lower than those that occur on the structures supported on a rigid base. For the cases when the fundamental period of the structure has values very different from the dominant ground period, soil-structure interaction leads in all cases to a reduction of the ductility demands, independently of the aspect ratio. For periods longer than the dominant ground period, this reduction results from the fact that soil-structure interaction takes the fundamental period of the structure to the descending branch of the spectrum, while for periods shorter than the dominant ground period this reduction can be explained in terms of the contribution of the kinematic interaction to the global system response. The reliability index β obtained from this information is a function of the base shear ratio and of the seismic intensity acting on the nonlinear systems subjected to the simulated motions. For the cases when soil-structure interaction is taken into account, this intensity depends also on the elongated natural period and on the effective damping of the soil-structure system. The resulting reliability functions are very similar for systems on rigid or on flexible foundation, provided that in the latter case the base rotation and the lateral displacement are removed from the total response of the system for the purpose of estimating effective ductility demands on the portion of the simplified reference system that represents the superstructure.

Figures 40 to 42 present a summary of a series of studies about the reliability functions of a set of hysteretic bilinear systems with a post-yielding stiffness equal to 0.05 of the initial tangent stiffness for linear response, located at three sites in the Valley of Mexico (SCT, CAO, D84). Figure 40 shows the response spectra that correspond to the mean values of simulated time histories, for different values of the ductility factor. Figure 41 shows the force-displacement relations of the structure 14S-3B resulting from the pushover analysis of the rigid-base structure, the corresponding fitted bilinear model and the fitted bilinear model for the system modified by the dynamic soil-structure interaction (DSSI).

For the earthquake scenario adopted (given values of magnitude and source-to-site distance, as described above), and the structural systems considered, effective intensities are significantly higher at SCT than at the other two sites, both for the rigid-base and the flexible-foundation systems, which leads to the lowest values of the reliability index calculated for this site. The resulting seismic reliability functions are shown in Figure 42.

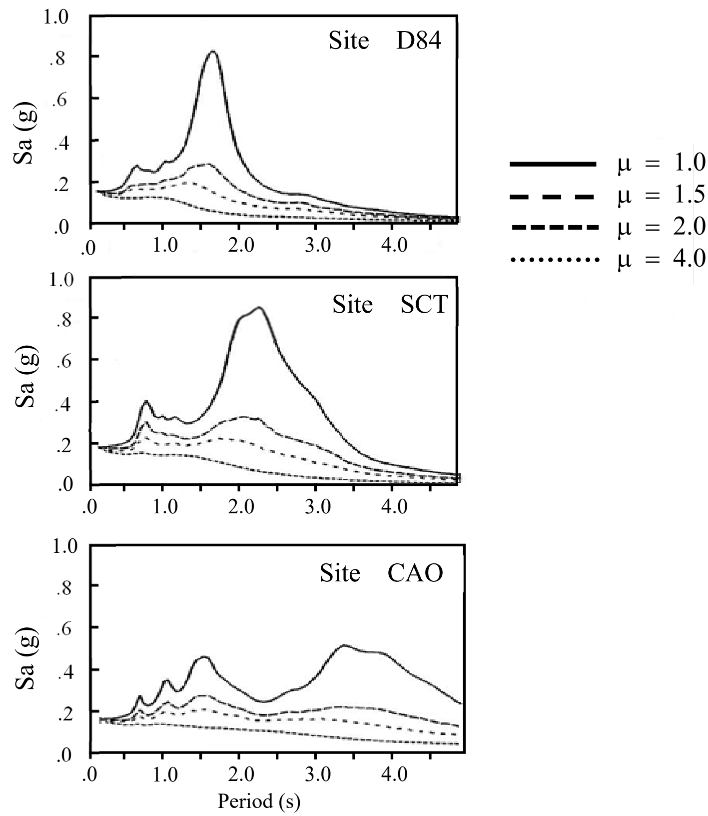


Figure 40. Response spectra that correspond to the mean values of simulated time histories, for different values of the ductility factor, μ.

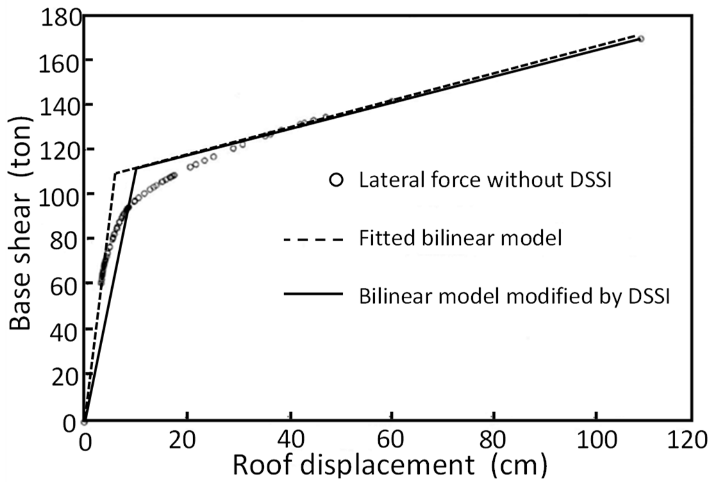


Figure 41. Force-displacement relation resulting from pushover analysis of the rigid- base structure, fitted bilinear model and fitted bilinear model modified by DSSI. Structure 14S-3B

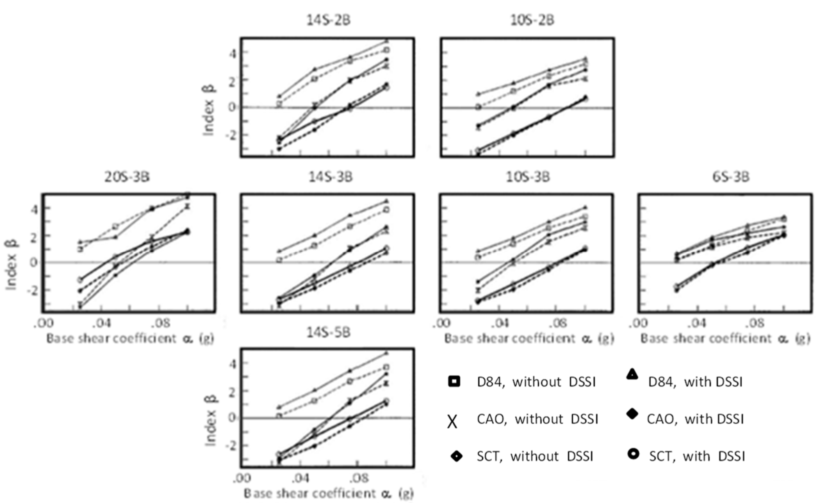


Figure 42. Reliability functions in terms of the base shear coefficient adopted for design at sites SCT, D84 and CAO.

3.3.4 Influence of damage accumulation

Seismic design criteria proposed for practical applications must be based on indicators about the expected performance of the system considered during a reference time interval. The expected performance is defined in terms of two indicators: the acceptable value of the probability of failure during the reference time interval and the present value of the expected costs of damage. Both are very sensitive to the evolution of the vulnerability of the system, produced by the accumulation of damage generated by the action of different permanent and accidental excitations, including, among others, dead and live loads, differential settlements, earthquakes and strong winds. Evaluating the evolution of the damage accumulate for a given system, at a particular site, is a very complex problem, because of the large uncertainties about the times of occurrence of the different excitations and their possible superposition; a practical solution may be achieved with the aid of Monte Carlo simulation.

Figure 43 shows some preliminary results of a study in progress with the support of the Institute for Structural Safety of Mexico City (Esteva and Díaz, 2019). In both graphs, the abscissa shows the value of the response amplitude indicator selected, corresponding to the first earthquake in the sequence, while the ordinate shows the indicator corresponding to the response to the second earthquake. The response amplitude indicators selected are $I_{SSR} = \frac{K_o - K_S}{K_o}$, where K_o is the initial tangent stiffness, K_S is the secant stiffness for the maximum lateral distortion of the system, and $\varphi = \frac{d_{Amax}}{H}$, where d_{Amax} is the maximum value of the relative lateral displacement of the top of the building with respect to its base, and *H* is the height of the building. Both indicators show a larger response for the second event; this is a consequence of the initial damage produced by the first event.

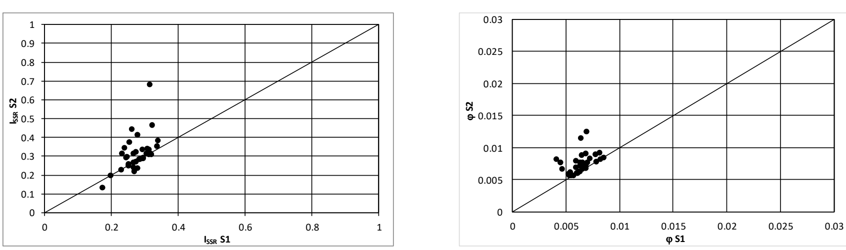


Figure 43. Response indicators obtained from results of dynamic response analysis of structures for intensities corresponding to a return interval of 125 years.

This preliminary information shows the significance of the damage accumulation process on the evolution of the seismic vulnerability of a structural system. It points at the importance of accounting for this concept in the formulation of reliability and performance-based seismic design criteria.

In order to take into account the influence of damage accumulation on the seismic vulnerability function of a system, Equation 7, presented in Section 3.3 must be replaced by the following¸ where d is a quantitative indicator of the damage that has been accumulated since the construction of the structure:

$$\overline{\delta}(y|d) = \overline{\delta}(y|S,d)[1 - p_F(y|d)] + \delta_F p_F(y|d) \tag{9}$$

3.3.5 Structural health monitoring

A very important action to take just after a structural system responds to the excitation produced by a high intensity earthquake is the evaluation of the damage that it has experienced, in order to make efficient decisions about its rehabilitation and reinforcement, in order to achieve an adequate performance during the rest of its expected life. However, evaluating the damage conditions in all the regions and members of the system by means of a visual inspection presents significant difficulties. This leads to the need of assessing its final conditions by means of an approach based on a comparison of its global and local stiffness properties for low intensity excitations; this constitutes the essence of the concept of "structural health monitoring". Esteva *et al* (2011) present a practical approach, based on Monte Carlo simulation, to estimate the expected performance function of a building for a specified time interval (expected life cycle), with decisions made using the information resulting from structural health monitoring. The following paragraphs present a brief description and an illustrative example about the application of this approach.

The global health condition is quantitatively expressed in terms of the index $I_{HG} = 1 - \Omega_f/\Omega_0$, where Ω_0 and Ω_f are, respectively, the fundamental natural frequencies of the system for the initial (undamaged) condition and for that resulting after the occurrence of a seismic excitation; this in is a function of the intensity of that excitation. The mentioned frequencies are determined with the aid of the transfer functions of the lateral displacements at the top and at the base, when responding to an environmental random noise excitation acting at the base.

The local health condition of the segment that extends from the base of the system to the top of the *i-th* story is expressed in terms of the index $I_{HL} = 1 - (\kappa_j)_{final}/(\kappa_j)_{initial}$, where κ_j is the ratio $\sigma_{Vj}/\sigma_{\delta j}$, of the standard deviations of $V_j(t)$ and $\delta_{ej}(t)$. Here, $V_j(t)$ is the shear force at the base, and $\delta_{ej}(t)$ is the lateral distortion of measured in terms of the relative displacement of the *i-th* floor with respect to the base. This means that the identification of local damage conditions is based on changes in the amplifications of story distortions with respect to the amplitudes of the motion at the base of the system (Esteva *et al*, 2014; Aldama *et al*, 2017).

For a multistory building, the expected value of the damage produced by an earthquake with intensity y is often expressed as a function of the expected value of the global distortion of the system, $\Psi = \Delta_N / H$. Here, H is the height of the building above the ground surface and Δ_N is the peak value of the relative lateral displacement of its top with respect to its base. In many cases, it results convenient to obtain the expected cost of damage as the sum of the contributions of several segments of the system, as given by Equation 10 (Esteva *et al*, 2002):

$$\bar{\delta}(y|S) = \lambda \sum_i \; r_{ci} \, \bar{g}(\Psi_i|y) \tag{10}$$

In this equation, $r_{ci} = C_{0i} / C_0$, C_{0i} is the initial cost of the i-th segment of the system, $g(\Psi_i)$ a function of the random value Ψ_i of the corresponding local distortion, and $\bar{g}(\Psi i|\ y)$ its expected value for intensity y. The initial costs C_0 and C_{0i}, as well as the joint probability density functions of the local distortions Ψ_i, are functions of the vector α of structural parameters. A factor λ, which is a function of the summation that follows it, accounts for the fact that repair costs include the contribution of a fixed amount that reflects the costs of the logistic arrangements necessary before the actual repair work starts. Because of this, λ will in general reach its maximum for infinitely small values of the summation mentioned above, and it will tend asymptotically to a smaller value as that summation grows. Esteva *et al* (2002) present more details about the determination of the expected damage function given by Equation (10).

In practical engineering applications related to performance-based earthquake-resistant design, the estimation of the failure probability of a nonlinear multi-story building is based on the concept of exceedance of the lateral deformation capacity of the system for a given value of the intensity of the seismic excitation; the lateral deformation capacity is determined by means of a conventional push-over analysis. However, probabilistic estimates of the deformation capacities of multi-story buildings obtained by means of push-over analysis are tied to severe limitations: it does not account for a) the influence of cumulative damage associated with the cyclic response, and b) the dependence of the lateral deformation capacity on the response configuration of the system when it approaches failure. This has fostered the development of alternative criteria, such as the incremental dynamic analysis (IDA, Vamvatsikos & Cornell, 2002), which permit the estimation of the seismic reliability function of the system without having to determine any deformation capacity.

Esteva and Ismael (2003), Esteva and Díaz-López (2006), Díaz-López and Esteva (2009), Esteva *et al* (2010) and Esteva *et al* (2011) presented a secant-stiffness-reduction index to be applied in the seismic reliability assessment of multi-story buildings. According to it, the reliability of the system under the action of an earthquake of known intensity but uncertain details about the ground-motion time history is expressed in terms of the probability density function of a secant-stiffness-reduction index (I_{SSR}):

$$I_{SSR} = \frac{K_0 - K_S}{K_0} \tag{11}$$

Here, K_0 is the initial tangent stiffness associated with the base-shear *vs* roof displacement curve resulting from pushover analysis and K_S is the secant stiffness (base shear divided by lateral roof displacement) when the lateral roof displacement reaches its maximum absolute value during the seismic response of the system. The failure condition is expressed as I_{SSR} = 1.0. For a given value of the intensity (y), the probability density function of $Q = \ln I_{SSR}$ is equal to $f_0(q)$, which is continuous for $q < 0$ and includes a discrete concentration at $q = 0$. This concentration is equal to $p_F(y) = P[Q = 0|y]$, the failure probability for an intensity equal to y. Esteva *et al* (2011) adopted this approach in an exploratory study about the influence of initial damage conditions on the damaging potential of new earthquakes.

The following paragraphs present an approach proposed by Esteva and Díaz-López (2006) and Esteva *et al* (2010) for the determination of the reliability function $\beta(y)$, starting from a sample of pairs of values of I_{SSR} and Y. Here, β is the safety index proposed by Cornell (1969) and y is a given value of the ground motion intensity. Assuming that, for an earthquake with intensity equal to y, the safety margin Z_M is equal to the natural logarithm of the ratio Y_F/y, the reliability function is the following:

$$\beta(y) = (E(Z_F) - lny)/\sigma(Z_F) \tag{12}$$

Here, $E(\cdot)$ and $\sigma(\cdot)$ stand for *expected value* and *standard deviation*, respectively.

For the structural system of interest, a sample of pairs of random values of Z and the stiffness reduction index, I_{SSR}, can be used to estimate means and standard deviations of $Z(u)$, the latter defined as the natural logarithm of the random intensity Y that corresponds to $I_{SSR} = u$. The values of Z in the sample that correspond to values of I_{SSR} equal to 1.0 are upper bounds of $Z_F = \ln Y_F$, where Y_F is the minimum value of Y required to produce collapse.

Esteva *et al* (2014) present an approach for the assessment of the influence of damage accumulation on the evolution of the seismic reliability function, using the information provided by the global and local damage indicators I_{HG} and I_{HL}. For illustrative purposes, they applied it to a ten-story symmetrical building structure, subjected to a sample of sequences of two single-component ground motion records, with the same intensity, in the direction parallel to the plan of symmetry of the system. The intensity of each record was measured by the ordinate of the 0.05 damping linear pseudo-acceleration response spectrum for the fundamental period of the system of interest.

The estimation of the seismic reliability functions for different conditions of initial damage was based on the responses of the system to two sets of sequences of simulated time histories of ground acceleration with intensities *"y"* comprised within an interval of values capable of generating different damage levels, from light to severe. The first set was oriented to the determination of the dynamic properties of the undamaged system and of their evolution resulting from damage accumulation. Those properties were measured by the transfer function of the spectral density of the ground acceleration to that of the displacement at the roof, relative to the ground. Each sequence consisted of a micro-tremor excitation, a seismic ground motion excitation and a second micro-tremor. The second set of sequences was intended to estimate the evolution of the dynamic properties of the system, starting from a condition with initial damage. For this case, each sequence consisted of an earthquake ground motion acting on the undamaged structure, a micro-tremor excitation, a second earthquake and a new micro-tremor excitation. The resulting structural response time histories were used to determine the values of D and of I_{HG}, as well as of the seismic reliability indexes for the system, including its initial undamaged condition and that corresponding to the intensity of the second earthquake, including the influence of the damage generated by the first one. In order to account for the uncertainties about the system properties (gravitational loads, mechanical properties of structural members), each sequence was applied to a sample of simulated values of the initial properties of the system.

Figure 43 shows a comparison of two response indicators (*ISSR*, φ) for a sample of two successive earthquakes with intensities corresponding to a return interval of 125 years. The influence of the damage accumulation in both response indicators is evident. Figures 44 and 45 show values of the peak lateral global distortion φ for two different buildings, for samples of pairs of two earthquakes with equal intensities, also corresponding to a return interval of 250 years.

The information presented above makes clear the need to develop seismic design criteria based on life-cycle optimization accounting for the influence of damage accumulation.

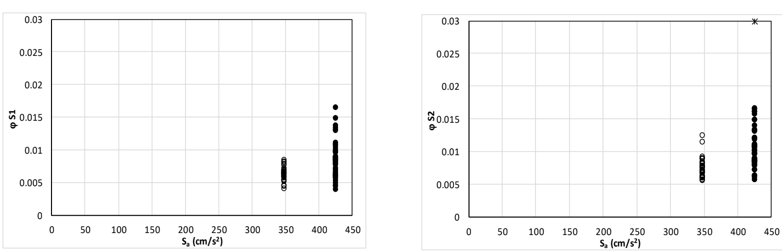


Figure 44. Values of the peak distortion φ for a sequence of two earthquakes with intensities corresponding to a return interval of 250 years (Ten-story buildings)

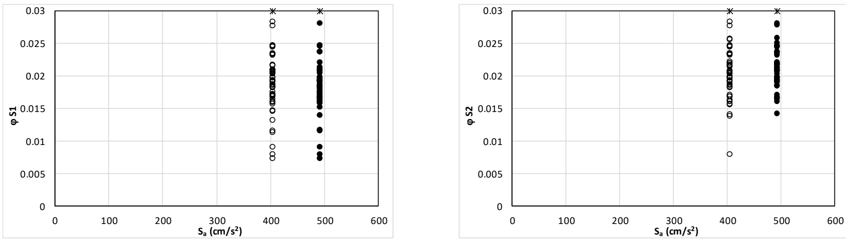


Figure 45. Values of the peak distortion φ for a sequence of two earthquakes with intensities corresponding to a return interval of 250 years (Seven-story building)

4 PERFORMANCE-BASED SEISMIC DESIGN CRITERIA

For a long time, life-cycle optimization has been recognized as an implicit and essential objective of engineering design, construction and maintenance actions. For the establishment of recommended practical criteria and methods for the achievement of that objective, it is necessary to adopt a formal decision framework, based on the identification and evaluation of an adequate set of quantitative indicators to describe the performance of a given system. In general, this description will be strongly affected by significant uncertainties, associated with a) the random variability of the times and intensities of future excitations and of the mechanical properties of the system, b) our imperfect knowledge about these concepts, and c) the limitations of the mathematical models used to represent them. This leads to the adoption of a probabilistic framework to describe the expected performance of a system; it should be capable of dealing with two types of uncertainties: random or aleatory (group a, above) and epistemic (groups b and c). In earthquake engineering problems, the excitations include the gravitational loads (dead and live) and the seismic events. The expected performance of a system when subjected to any of those events depends on both, the intensity of the latter and the mechanical properties of the former. Both variables depend on the level and distribution of damage that may have accumulated, because of the system's response to previous seismic events or of the action of any other agent, such as differential settlements due to gravitational loads.

4.1 Performance objectives and indicators

All types of damage costs and consequences produced by an earthquake on an engineering system have a strong correlation with the levels of physical damage that the system may experience when subjected to it, including the possibility of partial or total collapse. Therefore, determining the physical vulnerability function of a system, in terms of the criteria and methods adopted for its design and construction, constitutes the first step in the formulation of the corresponding life-cycle optimization analysis. An important consequence of damage accumulation is the increase of the ordinates of the seismic vulnerability function of the system. In order to maintain the resulting vulnerability levels within acceptable limits, specified design criteria, as well as maintenance and rehabilitation policies, must lead to the optimum solution that keeps failure probabilities within proposed acceptable levels. According to this approach, it is convenient to express the objectives of performance-based design criteria in terms of the following quantitative indicators:

- System reliability (probability of survival) for an earthquake intensity corresponding to a given return interval
- System reliability during a given life-time interval, taking into account the influence of damage accumulation, considering maintenance (repair and replacement) actions
- Life-time expected performance corresponding to the maximum utility function associated with a specified value of the life-time system reliability

4.2 Uniform hazard spectrum

A uniform hazard spectrum for a given site is a graph that presents in the abscissae the values of the natural period and in the ordinates the values of the pseudo-acceleration that have the same probability of exceedance per unit time at that site. As an example, Figure 46 presents the uniform hazard spectrum for a damping ratio of 0.05, at a soft-soil site in Mexico City, where the dominant ground motion period is equal to 2.0 seconds. It corresponds to a return interval of 125 years, which is equivalent of an expected annual exceedance rate of 0.008. Counting with this type of spectrum constitutes the first step in the development of practical seismic design criteria with quantitatively defined targets of seismic reliability levels and expected present values of utilities along the system useful lifetime.

Figure 46 also shows an envelope of the uniform hazard spectrum and a proposed design spectrum, with a simple format, adequate for engineering practice. This spectrum also intendeds to account for uncertainties about the mechanical properties of a structural system and about the gravitational loads acting on it.

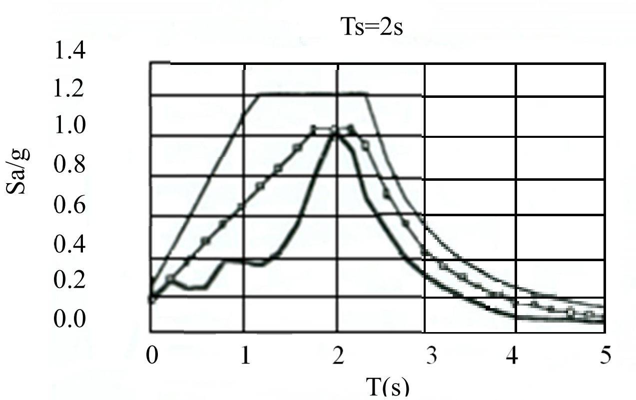


Figure 46. Uniform hazard spectrum for seismic intensities corresponding to a return interval of 125 years and proposed design spectrum

4.3 Development and calibration of consistent-reliability, practically applicable, seismic analysis and design criteria

Four alternative methods are presented in Section 3.1 for the estimation of the structural response amplitudes to be used for design; they range from the linear static approximation to the dynamic nonlinear step-by-step analysis. The response estimates resulting from any of the mentioned methods will include the influence of two sources of uncertainty. One of them, designated as aleatory, is associated with the random nature of two variables, namely the intensities and the detailed time histories of the ground motion excitations that may occur in the future and the statistical variability of the mechanical properties of the structural system of interest. As mentioned above, another type of uncertainty, designated as epistemic, arises from our imperfect knowledge about the statistical properties of the seismic process, or about the models used to represent the cyclic behaviour of the structural members or to estimate the dynamic response of the system (Jalayer & Cornell, 2003). The aleatory uncertainties are unavoidable, and are independent from the response-analysis methods adopted; however, the epistemic uncertainties are strongly dependent on them.

Performance-based seismic design criteria are focused on the control of the maximum values reached by specific quantitative indicators of structural response demands, such as peak lateral distortion amplitudes, dissipated energy or low-cycle fatigue indexes. Because of the uncertainties that affect them, these indicators can only be controlled within a probabilistic context. This calls the attention to the importance of developing rational criteria to account for these uncertainties in the formulation and calibration of practically oriented methods and algorithms for seismic analysis and design. For instance, the results presented in Figure 37 should be transformed into explicit functions expressing the value of $\bar{V}_y$ required to attain a specified value of β for the earthquake intensity S_a corresponding to a given return interval. Because functions of this type should cover a wide number of cases that are included in a given "family", the information required for their establishment must be developed through a parametric study, covering sufficiently wide ranges of values of all the variables that can have a significant influence on the values that may be reached by the performance indicators adopted.

For the illustrative example presented in the foregoing paragraph, the variable used to control performance (in this case measured by the reliability index β) is $\bar{V}_y$, the expected value of the base-shear strength, obtained by means of a pushover analysis. A more direct control of the expected performance target can be obtained if the normalized intensity is defined as $Z = S_d/\bar{\Psi}_c H$, where S_d is the ordinate of the displacement response spectrum for the design earthquake, $\bar{\Psi}_c$ is the lateral-distortion deformation capacity, determined in accordance with a specified conventional criterion, and H is the height of the building.

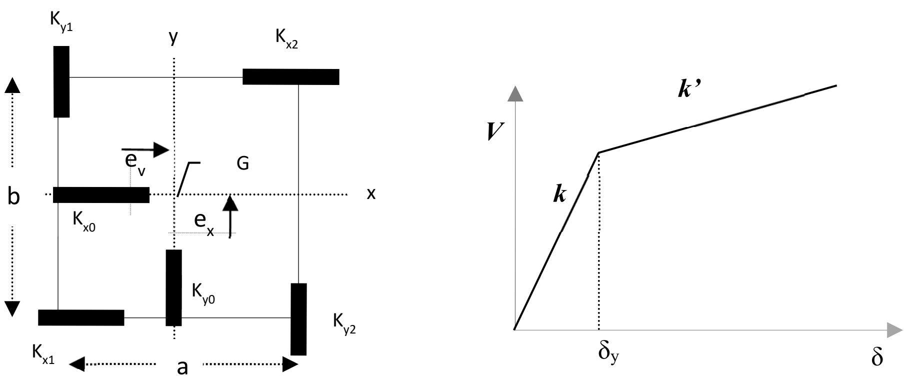
Other control variables can be adopted for the establishment of design acceptance criteria for the target performance level. Due to practical reasons, in the near future a large percentage of the urban constructions will be designed using the responses estimated by linear static or dynamic seismic response analysis. The safety factors that will be needed to cover the uncertainties associated with the response and performance predictions will be larger than those that would apply to cases studied with the aid of more refined criteria for dynamic response analysis. This concept should be clearly expressed in modern codified design recommendations.

For illustrative purposes, take for instance some preliminary results from an undergoing study oriented to the formulation of practical criteria for the evaluation of seismic vulnerability functions of multi-story buildings in Mexico City. One line of the study focuses its attention on in-plan asymmetric buildings, with different stiffness-and-strength torsional eccentricities. It starts from the fact that most available tools for the evaluation of the seismic vulnerability functions of buildings are limited for use with two-dimensional systems, or symmetrical systems with respect to a vertical plane, subjected to a single ground motion component contained in that plane. For this reason, it seemed convenient to develop some tools for the estimation of the seismic reliability functions of in-plan eccentric systems subjected to two simultaneous orthogonal ground motion components, by applying adequate corrective factors to the functions estimated for the two-dimensional or symmetrical systems mentioned above. The study was conducted with the aid of simplified single-story models, with a plan similar to that shown in Figure 47 a). The lateral strengths of the system in both directions are provided by a set of three elements in each direction, with bilinear shear *vs* deflection functions similar to that shown in Figure 47 b). For practical applications to multi-story buildings, these functions are estimated by means of a generalized version of pushover analysis, considering the action of two simultaneous orthogonal horizontal ground motion components; they are capable of representing the shear *vs* displacement of the system in each direction, its torsional stiffness and strength, and the eccentricities of stiffness and strength in each direction.

The results presented in Figure 48 correspond to a pair of systems similar to that shown in Figure 47a, with plan dimensions $a = b = 12$m, natural periods $T_x = T_y = 0.573$s, neglecting torsional vibrations, total mass = 261.57 ton·s^2/m (weight = 2566 ton), and a height $H = 30$m. Both the asymmetric and the corresponding symmetric system were designed for earthquake in accordance with Mexico City Building Code (NTCDS-DF, 2004), considering a distortion deformation capacity equal to $\Psi_C = 0.02$. For the asymmetric system, the following properties were considered: $r_x = r_y = 1.0$, $k_{x1}/k_{x2} = k_{y1}/k_{y2} = 0.5$, $\omega_y/\omega_x = 0.7$, $r = 1.5$. In these equations,

a) r_x and r_y represent the contributions of the edge elements to the initial stiffness of the system in directions x and y, respectively
b) ω_x and ω_y are the natural frequencies of the system in the x and y directions, respectively, neglecting the torsional-rotation degrees of freedom
c) $r = \Psi_{2yx}/\ \Psi_{1yx} = \Psi_{2yy}/\ \Psi_{1yy}$, where Ψ_{2yx}, Ψ_{1yx} are the yield distortions of edge elements 2 and 1 parallel to the x direction, Ψ_{2yy} and Ψ_{1yy} are the yield distortions of edge elements 2 and 1 parallel to the y direction (see Figure 47).

The strength and stiffness eccentricities in both directions are functions of these parameters.



a) b)

Figure 47. Simplified system used to represent a multistory in-plan asymmetric building: a) Plan, b) Shear-displacement functions of elements that provide the lateral strength

The reliability functions determined for both systems are shown by curves A and B in Figure 48, where I_S is the ground motion intensity, represented in this case by the quadratic mean of the ordinates of the pseudo-acceleration response spectra measured in two orthogonal directions, for the fundamental period of the system of interest. Curve C in the same figure was determined by means of a parametric analysis applied to a sample of 384 models with different values of the parameters defined in the foregoing paragraph. Because of the epistemic uncertainties introduced by the process of adjusting a function to a population of systems with different properties, the reliability levels corresponding to the function shown as curve C in Figure 48 are significantly lower than the values obtained by means of a direct reliability analysis for the system of interest.

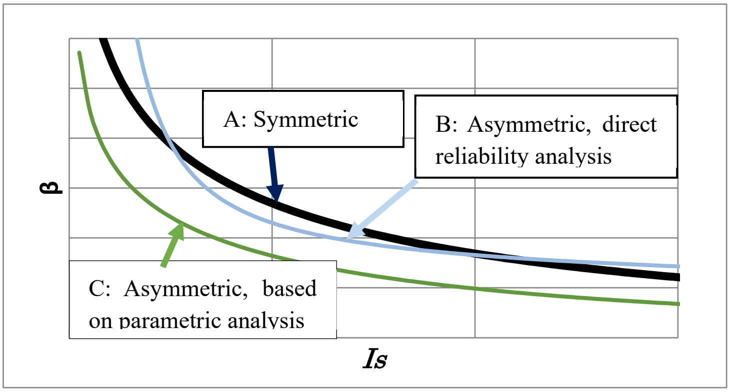


Figure 48. Seismic reliability functions for in-plan symmetric and asymmetric systems

- *Damage accumulation and life-cycle reliability analysis*

Ordinarily applied seismic design criteria explicitly accept the possibility of occurrence of different levels of structural damage for earthquake intensities associated with long or very long return intervals. Therefore, the process of damage accumulation must be taken into account for the establishment of seismic design criteria with specified reliability and expected performance targets within a life-cycle framework, not only for a single high intensity event. Formal criteria and tools to make engineering design decisions for systems exposed to infrequent accidental disturbances have been available for a long time (Rosenblueth, 1976); they have been applied assuming the occurrence of one single type of possible accidental disturbance (Esteva *et al*, 2010b). Only recently, the concept of "multi-risk" assessment has emerged, recognizing that the process of damage accumulation and the resulting expected failure rate are due to the superposition of the contributions of several types of accidental disturbances.

The fact that some "tolerable level of damage" must form part of engineering design criteria, specifically of earthquake-resistant design criteria, poses a new challenge, which goes well beyond the strict (although blind, in some cases) application of seismic design codes. This implies that the designer must select in advance the members, locations or critical sections where damage should be concentrated, having in mind at least the following: a) the occurrence of brittle failure mechanisms should be prevented, b) damage should be easy to detect and to repair. The use of hysteretic energy-dissipating devices constitutes an example of this type of risk control actions.

Accepting structural damage also implies that risk-based criteria must be developed and implemented to make decisions about acceptable damage levels, and optimum repair and maintenance policies. In the case of systems with hysteretic energy-dissipating devices, these policies must consider the need to replace each of those devices when it is estimated that it has reached a specified fraction of its expected fatigue life, even if there is no visual evidence of the level of accumulated damage. A life-cycle optimization model to establish these policies has been proposed by Esteva and Díaz-López (1993). In their formulation, these authors consider the interaction of the mentioned policies with the selection of the optimum seismic design requirements for the initial structure.

Differential settlements constitute another possible source of damage that may increase the seismic vulnerability of constructions. When significant, they must be taken into account, either in the design of new constructions or in the vulnerability and risk assessment of existing ones.

4.4 Design format and parameters

Section 4.3 presents a detailed explanation of the concepts that may have a significant influence on the dynamic response and performance of a structural system, including the large uncertainties that affect them. It also examines possible options to include this information in the formulation of seismic design criteria oriented to the attainment of pre-established levels of seismic reliability and expected performance. This information should serve as the basis of practically applicable design criteria, with simplified formats, as presented in the following, intended to achieve those levels.

- Linear response spectra, specified in a simple format (for instance, a plateau similar to that shown in Figure 46), covering the uniform hazard spectrum corresponding to the selected return interval of the intensity adopted for the purposes of the reliability targets described in Section 4.1. The width of this plateau intends to account for uncertainties about the dynamic response properties of the system considered; these properties result from the combination of the gravitational loads and of the mechanical properties of the members of the system. The example presented in Figure 46 corresponds to the 2004 version of the Mexico City seismic design code (Normas Técnicas Complementarias para Diseño por Sismo).
- Reduction factors (Q) intended to account for ductile-deformation capacity and over- strength of the system.
- Maximum lateral distortions γmax, for the design spectrum corresponding to the specified strength-reduction factor, Q.

4.5 Repair and maintenance strategies

In order to comply with the expected performance objectives mentioned in Section 4.1, it is necessary to adopt repair and maintenance strategies oriented to keeping the system reliability levels and the expected economic losses within acceptable limits during a given life-time interval. For this purpose, it is necessary to count with information about the evolution of the mechanical properties of the system, including the influence of damage accumulation, which may result from the actions of seismic excitations or from other types of accidental perturbations.

Structural health monitoring methods as those described in Section 3.3.5 are very useful for the generation of relevant information about the mechanical properties of a structural system at any instant; therefore, their application can serve to provide very significant information about the vulnerability function of the system after it responds to a high-intensity event, capable of generating significant damage. Making optimum repair and maintenance strategies imply determining acceptable levels of reduction in the ordinates of the seismic reliability functions and of increase in the expected damage functions for given intensities and, therefore, in the present value of the expected losses during the system life-time. Esteva *et al* (2014) propose the following approach:

a) Adopt a the following equation to represent the expected present value of the life-cycle utility of the system of interest:

$$U = C_0 + E[\sum D_i e^{-\gamma T_i}] \tag{13}$$

b) Define an adequate structural health indicator, intended to account for the influence of cumulative damage on the global properties of the system; for instance, I_{Hi} = 1 - Ω_{0i}/Ω_{00}, where Ω_{00} and Ω_{0i} are respectively the dominant natural frequencies of the undamaged system and of that corresponding to the damaged condition after the *i-th* seismic excitation (Esteva *et al*, 2014).
c) Consider several alternatives of threshold I_{Hi} values of the structural health indicator that can be accepted before repairing the system, restoring its mechanical properties to those corresponding to the undamaged condition
d) For each of those alternatives, perform a Monte Carlo simulation of possible times of occurrence and intensities of seismic events
e) Perform a repair action consistent with the strategy selected
f) Generate a sample of simulated structures, assuming zero-initial damage conditions
g) Generate a new sample of pairs of combinations of ground motion histories and properties of structural systems.
h) Update the seismic reliability function after each event, including the influence of damage accumulation and of the repair action performed.
i) Obtain the expected value of *U* for the strategy selected.
j) Select the repair strategy leading to the minimum expected value of *U*

5 SEISMIC-RISK-REDUCTION PROGRAMS

5.1 Seismic vulnerability and risk assessment:

Engineers and groups in charge of civil protection programs agree about the advisability of implementing seismic retrofit programs oriented to controlling risks in urban constructions. Efforts should focus on those constructions that present the highest expected values of human, economic and social losses; these include the following, among others:

a) Schools, auditoriums, theatres, and others that may congregate large numbers of persons,
b) Hospitals, transportation terminals, firemen and police stations, tele-communication centres, and others whose operation is essential during an emergency situation,
c) Constructions containing explosive or highly flammable or toxic materials,
d) Museums, public register offices, and buildings that contain specially important and non-replaceable materials, such as works of art, elements of social heritage, files necessary for the functioning of the society, etc.

In order for these programs to be as efficient as possible, they must include, at least, the following actions:

a) Preliminary identification of high-risk constructions (sidewalk evaluation criteria)
b) Vulnerability and risk assessment with the aid of simplified models
c) Detailed vulnerability and risk analysis

Each action in this process generates information useful for the identification of the constructions that deserve study in the next action. Several general and detailed guidelines have been developed around the world for the implementation of seismic-risk-reduction programs; to name just a few: ASCE (2003), JBDPA (2001a-c), Kuroiwa (2004). An overview of several concepts and programs oriented to the control of seismic risk in school buildings is presented in OECD (2004).

5.2 Ensuring code compliance and quality control

Counting with modern, advanced, practical and robust building codes and technical norms is not a guarantee of adequate reliability levels. Lessons learned from the observed earthquake performance of engineering works go beyond the technical aspects of that discipline. We have also learned that a large number of cases of seismic deficient performance or collapse is associated with human errors or wrong attitudes such as careless design, faulty workmanship and deficient quality control, which in turn may be associated with professional incompetence or limited risk consciousness. Seismic damage prevention and risk control strategies require, therefore, the implementation of combined programs that include state-of-the-art knowledge and tools, continuous education and professional updating programs, and policies intended to ensure code-compliance practices. The latter may include, for instance, peer-review programs of the structural design documents and of the construction and quality control processes of all essential facilities and constructions whose failure can produce large human, social or economic losses, and a number of randomly selected constructions of other types.

5.3 Non-engineered construction

A large percentage of the human losses generated by past earthquakes has been associated with the collapse of non-engineered constructions: houses and buildings erected using local materials and traditional structural arrangements, which have not benefitted from the great advances in knowledge and technical resources attained by earthquake engineering during the last few decades. The causes of the high vulnerability of non-engineered constructions are well known, but their elimination or control are extremely difficult, because of the socio-economic constraints that do not permit the adoption of sufficiently high safety level standards for the constructions housing large portions of populations in developing countries. In spite of these limitations, the present state of research indicates that reasonable levels of structural safety can be achieved by adopting appropriate design and construction details involving only small extra expenditure, which should be within the reach of people in most countries (IAEE-NICEE, 2004). The challenge stands of developing dissemination and training programs capable of transmitting these techniques, together with the necessary risk awareness and risk-reduction attitudes, to the social groups that require them. Unfortunately, in many developing countries, making the essential economic and technical resources available to the least favoured members of society is still a significant challenge.

6 CONCLUDING REMARKS

The evolution of Earthquake Engineering has been the combined result of both, lessons learned from earthquakes and from conceptual engineering models. The latter have often been developed after painful experiences suffered, resulting from the occurrence of highly damaging seismic events. Many of these experiences would have been avoided if we, engineers, authorities and inhabitants of the zones exposed to earthquake hazard, had joined forces to create awareness and enhance consciousness about the magnitude of the risk and the means and tools to mitigate it.

During the last few decades, the progress of Earthquake Engineering has been spectacular:

- We have greatly improved our understanding of the sources of seismic hazard, of the processes of earthquake generation and propagation and of the cyclic behavior of structural members and systems that determine the dynamic response of complex nonlinear systems to high intensity seismic excitations;
- We have developed complex and powerful mathematical models and computational tools to apply this knowledge.

Unfortunately, in many cases we have failed to identify in advance previously ignored potential sources of seismic hazard and risk. We have not devoted enough efforts to produce simple-to-apply engineering design criteria and tools, aimed at attaining previously specified reliability and expected performance targets; and, finally, we have not been able to create enough conscience among the engineering community about the need to understand the uncertainties that lie beyond the simplified methods ordinarily applied in the practice of earthquake engineering.

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