2.3 EARTHQUAKE ENGINEERING AND SEISMIC RISK

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Introduction

Earthquakes are one of the most dangerous and destructive natural hazards. They strike with no warning and can devastate entire cities. In the past century over 2 million people have died due to earthquakes and earthquake related disasters (USGS). Recent events in modern cities have also illustrated the devastating effects earthquakes can have on economies (losses from the Kobe 1995 earthquake in Japan are estimated at over US$100 billion, EQE International). Seismic Risk is defined as: the likelihood of losses from earthquakes in terms of deaths, injuries, economic losses, damage to buildings and infrastructure. However, it is the damage of buildings, industrial plant and infrastructure that is the major cause of the other losses (to life and economic) associated with earthquakes and not the ground motion itself.

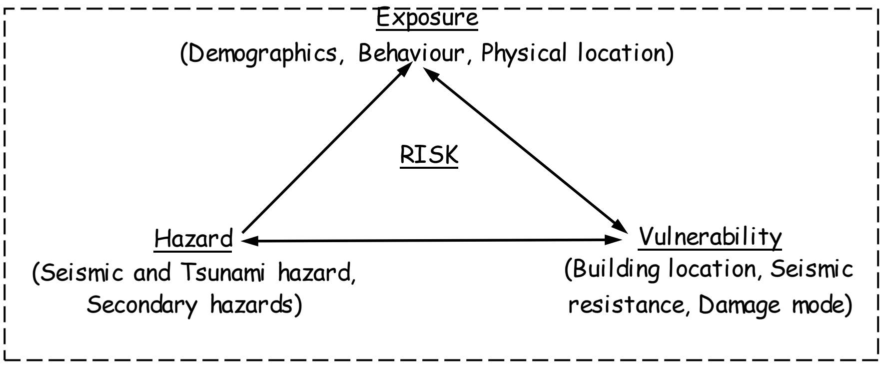
In the field of Earthquake Engineering seismic risk assessments are carried out on populations of buildings to identify the urban areas most likely to undergo large life and economic losses during an earthquake. The results of such studies are important in the mitigation of losses under future seismic events, as they allow decisions to be made regarding building strengthening and allow disaster management plans to be drawn. Seismic risk evaluation is also a key element in the development of seismic design codes for new constructions. The main design problem is that an intense earthquake usually constitutes the most severe loading to which most civil engineering structures might be subjected, and yet in most parts of the world, even those that are highly seismic, there is a possibility that an earthquake may not occur during the life of the structure. Hence in seismic design, structures are not designed to resist the applied earthquake loads elastically but are allowed to enter their non-linear range of response, which means that they are designed to sustain a certain level of damage. Capacity design concepts were introduced in seismic codes in the 1980s. These set out criteria for structural design that ensure the location and severity of the damage in the structure is controlled and does not pose a risk to life, (even though the structure may need to be pulled-down following the event). This approach to design is not 'fail-safe' but can be termed 'safe–to-fail' as referred to by Snowden (see Section 2.10 in this same Working Paper). The acceptability of a certain level of damage occurring depends on the importance and use of the structure and the seismicity of the area. The fundamental issue is to seek to balance the cost of providing earthquake resistance with the losses that this investment will prevent. A seismic risk evaluation is therefore required in order to guide decisions as to acceptable performance and the definition of design criteria to achieve these objectives. These issues are explored in more detail later in this paper, where new paradigms in seismic design that allow explicit consideration of seismic risk are discussed.

The seismic risk assessment model is often considered to be composed of three elements:

- ***Hazard:*** The sum of seismic, tsunami and secondary hazards. It is commonly expressed as the annual probability of different severities of earthquake. It cannot be reduced, as it is determined by Nature.
- ***Vulnerability:*** This consists in the assessment of the susceptibility to damage of the local building inventory. Vulnerability is man-made and can be reduced through seismic design and construction (or through strengthening of existing buildings).
- ***Exposure:*** Exposure can mean the type and number of buildings, their location, number, type, occupancy and value. Exposure evaluation is the forecast of human and economic losses from the predicted damage scenario using exposure data.

Complex interactions exist between the formation of earthquakes, the resulting ground motions, the response of infrastructure to the ground shaking, the resulting infrastructure damage, human injuries, fatalities, direct and indirect economic losses (e.g. see Figure 1). However, in practice the seismic hazard assessment, vulnerability and exposure evaluations are carried out separately and brought together only in the final risk evaluation.

Figure 1: A visualisation of seismic risk evaluation



Note: The arrows indicate interactions between the main factors
Source: Bommer,1998

Uncertainties are associated with each constituent part of the seismic risk paradigm (i.e. hazard, vulnerability and exposure). These uncertainties are large and can significantly impact the risk assessment and consequent decision making process (Grossi *et al.*, 1999). These uncertainties can be grouped into the following generic categories:

- ***Inherent variability*** in the earthquake and its effects (Alleatoric uncertainty)
- ***Limited knowledge*** of the phenomenon (Epistemic uncertainty)
- ***Inability to accurately represent/model*** what we know (Paradigmatic or Modelling uncertainty)
- ***Differences between assumed and actual values*** (Parametric uncertainty)

The main way of quantifying aleatoric uncertainties is by using observations from past earthquake events or from experiments. Commonly epistemic uncertainties are also dealt with by using empirical approximations. However in the case of the new science of earthquake engineering, which deals with the characterisation of a rare hazard, observational data is limited in quantity. Machines for recording earthquakes (seismographs and accelerographs) have only been around for approximately 100 years, which is a short timeframe when you consider that large earthquakes have recurrence period that can be several hundred years. Large earthquakes do not always occur near urbanised areas, hence there is a lack of knowledge of the response of different buildings to ground shaking. Furthermore, large-scale experiments for measuring building response to earthquake loading in a laboratory are few due to their expense in terms of time and money. Uncertainties associated with the seismic risk assessment of an urban area may be further exacerbated by, the lack of available data regarding local inventory, building use and local ground conditions (e.g. see Table 1). However, component uncertainties are often inappropriately represented, rarely combined and often ignored in the risk assessment.

Table 1: Comparison of the data required and that typically available for a seismic risk assessment

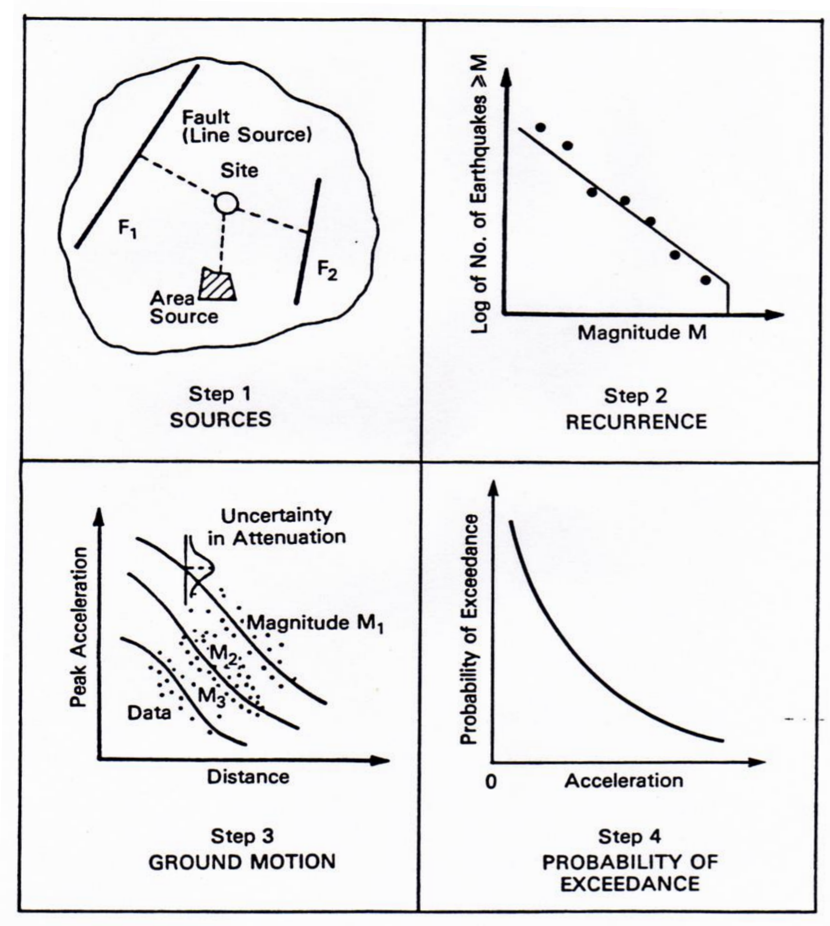
What do I need to know?	What do I know?
Inventory data : ▫ structure locations ▫ structure types ▫ configurations ▫ materials ▫ heights ▫ detailed design data.	Inventory data: ▫ number of structures ▫ predominant structure types ▫ number of stories ▫ rough guess at seismic design level
Soil type data ▫ depth of strata ▫ shear wave velocity ▫ liquefiability	Soil type data: ▫ geological maps ▫ a couple of bore holes ▫ a hazard map
Typical ground motions for local sources of earthquakes for a range of different event magnitudes	A few records recorded far from the site, if lucky.
Understanding of local building dynamic responses and typical failure/damage modes	Some past earthquake observations of building response.

Priestley (1998) showed that the choices made for the analysis method, structural idealisation, seismic hazard and damage models can lead to significant discrepancies in seismic risk assessments made by different authorities for the same location, structure type and seismicity. To better understand why such variations in seismic risk assessments occur, within the following sections of this paper each stage of the risk evaluation process is summarised, the main sources of uncertainty are highlighted and their treatment in practice is discussed.

Seismic Hazard Assessment

Earthquakes occur when stresses in the Earth's crust (due to the relative displacement of the tectonic plates) exceed the strength of the rocks causing the crust to rupture along lines of weakness (faults). Until the moment of rupture energy is accumulated at the fault and mainly stored as elastic deformation. When rupture occurs this energy is suddenly released and propagates out from the fault in the form of seismic waves. The seismic waves induce violent shaking of the ground often felt up to hundreds of kilometres away from the earthquake source and can cause severe damage to the built environment. The aim of a seismic hazard assessment is to determine the size and characteristics of the ground shaking at the site where the risk is being assessed. 'Determination' is an optimistic term in this context as earthquakes cannot be accurately predicted. Although the location of major faults and tectonic plate boundaries can be identified from observation of past earthquake events, it is not yet possible to predict when and where the next earthquake will occur, nor how large it will be. Seismic hazard is therefore typically evaluated using statistical methods, which look to extrapolate the pattern of previous earthquake occurrences into the future.

Figure 2: The four main steps in a PSHA



Source: Reiter, 1990

In general, a Probabilistic Seismic Hazard Assessment (PSHA) consists of the following steps (Bommer 1998), which are illustrated graphically in Figure 2:

- Definition of the nature and locations of earthquake sources;
- Determination of magnitude-frequency relationships for the sources;
- Choice of ground motion prediction (attenuation) equation to estimate the ground-motion at distances from the sources;
- Evaluation of the ground motion at a site and associated exceedence probability.

The first step involves the identification of all major tectonic and geological structures and all major faults in the vicinity of the area to be assessed. Next, instrumental earthquake data for the area is collected from earthquake catalogues and from historical sources. The earthquake data is plotted over the map of the identified geological structures and the two are correlated (i.e. the earthquakes caused by the known tectonic features and faults are identified). Where the seismicity is not characterised by a well-known and defined fault, seismic source areas (zones) are defined. Seismic source zones represent areas where a uniform tectonic regime is assumed to exists, and where earthquakes are assumed to have an equal probability of occurring anywhere within the zone. The choice of the size and shape of seismic sources is subjective but variations in these has been shown to have a large effect on the seismic hazard (Barbano *et al.*, 1989).

To characterise the seismicity of any source zone, a catalogue of events associated with the source is extracted from the regional catalogue. Within a seismic zone a simple linear relationship can often be found between the log of the annual frequency of occurrence of earthquake events and their magnitude. This relationship is termed ***recurrence relationship*** and was characterised by Gutenberg and Richter (1954) as:

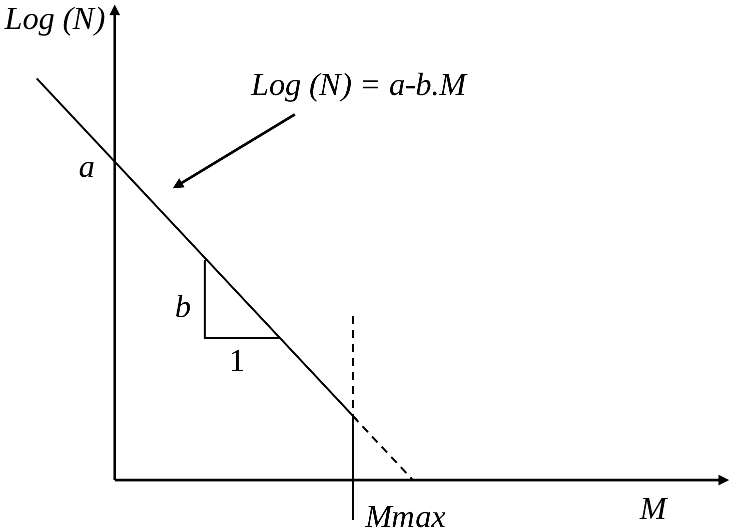
$$\log(N) = a - b.M \qquad \textbf{[1]}$$

where N is the number of earthquakes per year with magnitude greater or equal to a given size, M. a represents the level of earthquake activity. b reflects the relative number of small and large earthquakes, and takes values between 0.5 and 1.5 (in many parts of the world b will often be close to 1.0). Maximum (M_{max}) and minimum (M_{min}) bounds need to be determined for the regression. This also involves some degree of judgement. M_{max} can either be inferred from known fault dimensions and an established relationship between fault rupture and magnitude, or else through the addition of a small increment of magnitude to the largest earthquakes historically known to have happened in the area. M_{min} instead defines the level below which the earthquake catalogue is incomplete (not all small earthquakes were recorded, especially in the past when less sensitive accelerographs existed).

One of the main problems faced by the engineer or seismologist is the lack of a complete database of earthquakes for any tectonic region, and the brevity of the recording period compared to the recurrence rate of large seismic events. Studies have been carried out in many regions to evaluate historical earthquakes from old literature sources (e.g. newspapers). Important such studies include those by Ambraseys and

Melville (1982) and Ambraseys *et al.* (1994) for Iran, Egypt, Arabia and the Red Sea. Paleoseismology and has also contributed to the pool of knowledge by dating seismic events from observations of fault offsets and geological studies.

Figure 3: Earthquake recurrence relationship



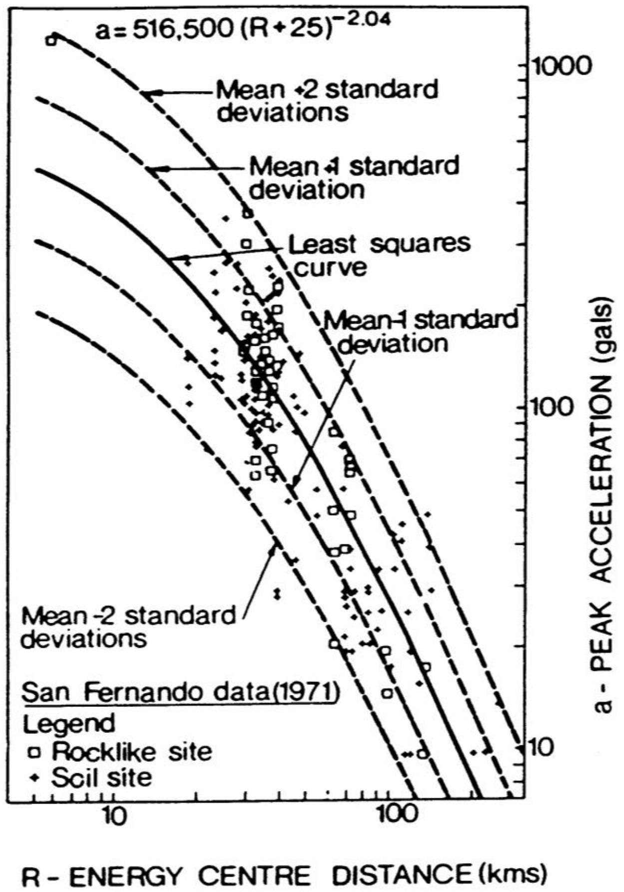
Source: Bommer, 1998

The third stage of a seismic hazard analysis involves deriving the likely strong ground motion in the study area, given the seismic source zones and their associated recurrence relationships are known. This is done via the use of ground motion prediction (or attenuation) relationships. Attenuation relationships effectively model how a strong ground motion parameter (e.g. peak ground acceleration) changes with differences in earthquake source, travel path taken by the seismic waves between the source and the site at which the strong-ground motion is to be predicted, and the characteristics of the soil at the site itself. They are empirical relationships derived from databases of accelerograms from past earthquakes. Many such relationships exist, as summarised in Ambraseys and Bommer (1995) and Douglas (2003). Different attenuation relationships exist for different countries and types of seismic sources. They commonly take the form:

$$\log(pga) = c_1 + c_2 M + c_3 \log(r) + c_4(r) + \sigma P \quad \textbf{[2]}$$

where r is defined by $r^2=d^2+h_0^2$, d is the horizontal source-site distance and h_0, c_1, c_2, c_3 and c_4 are determined from the regression, h_0 representing the depth of the earthquake focus. There are two terms representing distance within the equation because there are two ways in which energy is dissipated with distance from the source. The first term represents the geometrical spreading of the seismic wave front and the second term represents the inelastic attenuation due to energy absorption. It must be realised that these attenuation relationships are very simple representations of the very complex processes occurring when seismic waves travel through the ground. Hence all attenuation relationships have an associated value of uncertainty, the last parameter in equation 2. σ is the standard deviation of the relationship, with P taking a value of zero if the mean value of pga is required or 1 if the mean + one standard deviation (i.e. the 84-percentile) value of pga is needed. Graphical examples of attenuation relationships are given in Figure 4.

Figure 4: Mean and standard deviation plots of attenuation relationships regressed from pga data from the San Fernando (California) earthquake of 9 February 1971



Source: Dowrick, 1987

Once an appropriate attenuation relationship has been selected for use in the assessed location, the seismic hazard the site can be evaluated. The seismic hazard is typically expressed in terms of a probability of exceedence, q . q is the probability that an earthquake of magnitude M or greater occurs during a given time span L (years)

$$q(pga) = 1 - e^{-L\sum_{i=1}^{n} N_{pga,i}} \quad \text{where} \quad N_{pga,i} = 10^{(a_i - b_i . M_{pga,i})} = \frac{1}{T_{pga,i}} \qquad \textbf{[3]}$$

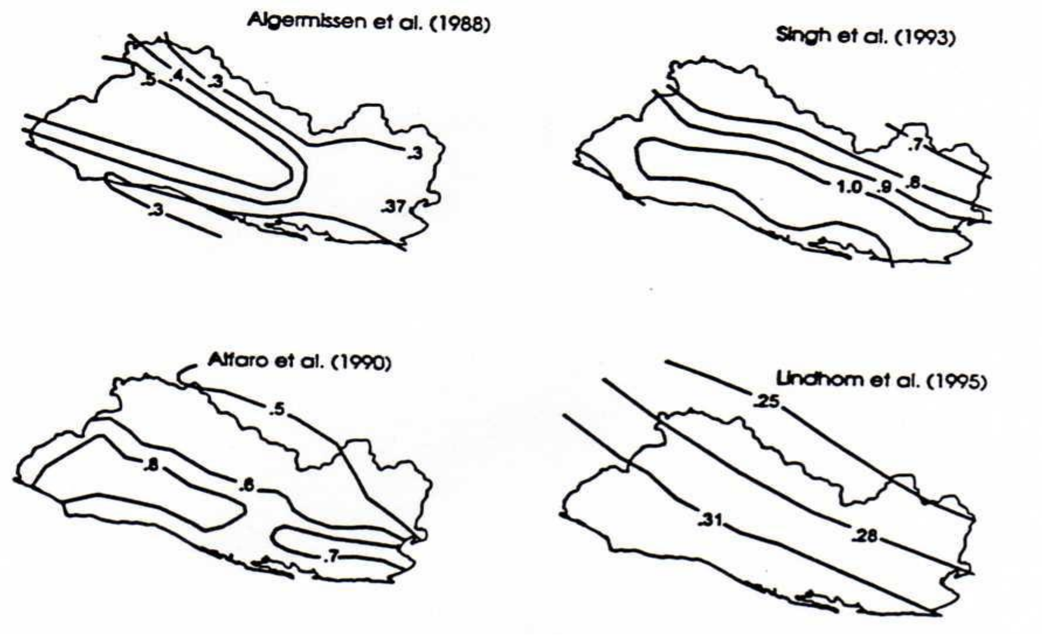
where n is the number of earthquake sources that can cause a given ground motion parameter size (pga) at the assessed site. $M_{pga,i}$ is the minimum size of earthquake generated at earthquake source i that can produce the selected pga at the site (as determined by the attenuation relationship). $N_{pga,i}$ is the associated annual frequency of occurrence of $M_{pga,i}$, as determined from the recurrence relationship for the earthquake source. $T_{pga,i}$ is defined as the return period of the earthquake of magnitude $M_{pga,i}$. The results of a PSHA, including uncertainties, can be represented as a series of curves (mean, median, or selected fractiles), showing the annual frequency of exceeding different levels of the chosen measure of ground motion (SSHAC, 1997).

The exceedence probability expression assumes earthquakes follow a Poisson process (probability distribution), which in turn assumes that the probability of having a favourable event in each trial is small compared to the number of trials, and that the process is stationary in time, i.e. that the probability of a favourable event occurring is the same in all trials and that events are independent. What do these assumptions mean when we talk of the probability of earthquake occurrence? If each trial is assumed to be a year in the life of the building, then the first assumption states that the probability of an earthquake occurring with magnitude equal or greater than M in one year is small compared to the number of years in the design life of the structure, L. This is true as earthquakes are rare events, and so the assumption is adequate. The second assumption instead implies that the probability of an earthquake with magnitude greater or equal to M occurring in any year is the same, irrespective of whether one such earthquake has or has not happened in recent history. This assumption means that in modelling the occurrence of earthquakes using the Poisson process we are assuming the process of earthquake formation has no memory. In practice this implies that at a given site an earthquake has the same likelihood of occurring today as it did yesterday, even if yesterday a large earthquake occurred at the site. This is physically incompatible with our understanding of how earthquakes are formed (i.e. faults rupturing and releasing energy that has built up over time at the fault/tectonic boundaries). In this formulation, faults are also assumed to be independent, i.e. the rupture in one fault, or in one portion of a fault will not trigger the rupture of another located nearby. However, many are moving towards a belief that the earth's crust in tectonic zones is in a state of 'self-organised criticality' (Turcotte, 1991) and have suggested using a dynamical systems approach to the prediction of earthquakes. However, these methods have not fully been researched or accepted in the seismological field. The Poisson process can be considered acceptable if we are evaluating the hazard for any general period of exposure and are not considering the time of occurrence of the last earthquake.

In most seismic codes of practice the design seismic ground motions for residential buildings are associated with a 10% probability of exceedence in 50 years. This implies a value of N of 0.0021 and a return period, T, of 475 years. For more important structures this probability of exceedence is reduced, (i.e. the return period of the design earthquake event is increased). The PSHA in codes of practice is provided in the form of a seismic hazard map. This summarises the ground motion values associated with a chosen exceedence probability. The results of PSHA can be heavily influenced by uncertainties introduced by the quantity and quality of earthquake data used to determine the recurrence relationships, by the subjectivity of the definition of seismic source zones, the choice of maximum and minimum earthquakes for a source and by the selection of attenuation relationship (SSHAC, 1997; Bernreuter *et al.*, 1987; Ellingwood, 2001). This is illustrated in Figure 5 by the different hazard maps produced by different researchers for the same area and considering the same return period.

The derivation engineering parameters to represent the likely strong ground motion is a further area where uncertainties arise. An accelerogram (earthquake trace of acceleration versus time) is the most complete representation of earthquake ground motion. The damage potential of earthquake time-histories varies according to their characteristics such as the number of load cycles, amplitude and duration of the ground motion, and the relationship between record frequency content and the dynamic characteristics of the assessed buildings. In order to carry out an engineering assessment or design, it is necessary to represent the accelerogram by a single index or parameter. Many parameters representing a single or multiple characteristics have been proposed for this purpose (Clough and Penzien, 1975; Nau and Hall, 1984; Araya and Saragoni, 1984; Benedetti *et al.*, 2001). Different degrees of success are achieved, but none manage to account for all the characteristics that affect building damage potential and most require the earthquake record for the site to be known prior to their evaluation, which is not practical in a predictive context. In view of the above discussion it is not surprising that the Seismic Hazard is the largest source of uncertainty in the Seismic Risk evaluation (Wen, 2001).

Figure 5: Hazard maps for El Salvador (Central America) showing accelerations (g) with 475-year return period determined by four independent studies



Source: Bommer et al., 1996

Vulnerability Evaluation

In developed countries 'satisfactory' or 'modern' codes for seismic hazard-resistance were only really introduced in the late 1970s and 1980s. Thus, buildings and infrastructure built prior to this (i.e. most of the existing building stock!) pose a risk. However, even non-seismically designed buildings have an inherent resistance to earthquake loads, which may be sufficient to resist the hazard in low or medium seismicity areas. The aim of a vulnerability evaluation is to asses whether this is the case, and if not what the likely state of damage will be. Such an evaluation can inform the decision-making process for prioritising areas for intervention (e.g. via structural strengthening). Vulnerability (or fragility) curves can be derived that provide relationships between ground motion and the probability of exceedence of certain thresholds of damage. In mathematical terms, the probability of reaching or exceeding damage state d_i given that the ground motion level is gm_k, is given by:

$$P_{ik} = P[D \geq d_i \mid GM = gm_k] = \sum_{j=i}^{n} P[D = d_j \mid GM = gm_k] \qquad \textbf{[4]}$$

If P_{ik} is evaluated by varying k, (i.e. the ground motion severity), whilst keeping i constant, a vulnerability curve is obtained for the damage state i. The vulnerability curve shape is dependent on the construction material and lateral load resisting system of the structure. This means that several vulnerability curves are required to evaluate the vulnerability of a city. Vulnerability curves (VC) for building populations are constructed from post-earthquake damage statistics which derive from:

- Post-earthquake surveys - Empirical VC
- Expert opinion – Judgement-based VC
- Analysis of sets of building models under increasing ground motion severities – Analytical VC
- A combination of sources – Hybrid VC

The reliability and usefulness of the vulnerability relationships depend on the statistical data source, the derivation methodology and the chosen ground motion parameter and damage scale. The observational source is the most realistic as all practical details of the exposed stock are taken into consideration alongside soil–structure interaction effects, topography, site, path and source characteristics. However, in deriving empirical vulnerability curves it is assumed that the earthquake damage observed in buildings in the past is representative of the future performance of similar constructions, subjected to comparable events. This may not be true if the building stock has significantly changed, for example if there have been substantial repairs or upgrading. This source of data is also severely limited. Few earthquakes occur near densely populated areas, hence the data is scarce and highly clustered in the low-damage, low-ground motion severity range. The quality of available survey data is also of concern. Surveys are not always carried out by trained engineers, or may not cover a statistically valid sample size. Moreover, damage due to multiple earthquakes may be aggregated and attributed to a single event or buildings damaged as a consequence of phenomena other than ground shaking (e.g. ground subsidence, landslides, flooding and fire) included in the data. Rarely do post-earthquake surveys distinguish between buildings of different materials, heights or seismic design provisions. Consequently the curves are highly specific to a particular seismo-tectonic, geotechnical and built-environment, and are unreliable due to the survey data scatter (Orsini, 1999). The correlation of the vulnerability curves with the observed post-earthquake damage data used to derive them ranges between $R^2 = 0.4$ to 0.6 (Rossetto and Elnashai, 2003).

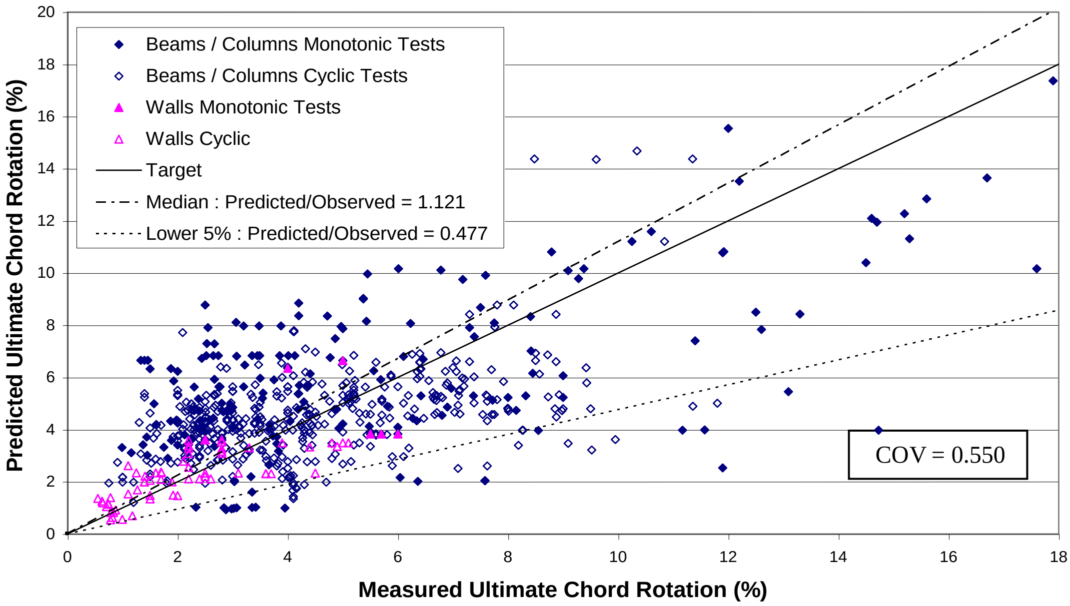
Judgement-based curves are derived from damage statistics derived from the opinion of experts. A common method used is to ask earthquake engineering experts to give estimates of the probable damage distribution within building populations when subjected to earthquakes of different sizes. For each earthquake size, probability distribution functions representing the range of damage estimates can be fit to the expert predictions. The probability of a specified damage state can then be obtained from these distributions and plotted against the corresponding ground-motion level to obtain a set of vulnerability curves and associated uncertainty bounds. Expert opinion is an unlimited source as experts can be asked to provide damage estimates for any number of structural types. Consequently, it is the predominant source for vulnerability curves found in most rehabilitation codes (e.g. ATC-40). The choice of experts, the method of collection and aggregation of their opinions is crucial to the reliability of the curves (Paté-Cornell, 2002). For example, SSHAC (1997) found that large differences in the results of their hazard assessment were obtained when the expert's opinions were treated as 'noisy observations' of the quantities of interest compared to when each expert's opinion was weighted by a factor dependent on their confidence in their estimate. Bias may exist amongst the experts and it is almost impossible to assess the conservatism inherent in the expert's opinions. Hence, unless the curves derived in this way are validated with observational or experimental data the reliability of judgement-based curves is questionable.

Analytical vulnerability curves are derived from damage statistics generated from the analysis of sets of building models under increasing ground motion severities. Many such curves have been proposed by researchers for different building types (Singhal and Kiremidjian, 1997). However, this source has not been used to the limits of its potential. The derivation process is computationally intensive as many analyses are required in order to represent the large variations in the ground motion and the capacity of buildings within a population and provide statistics at several earthquake intensities. For example, coefficients of variation in building resistance of up to 40% have been reported by Sues *et al.* (1985) and Cornell (1996). These variations in structural capacity are due to differences in configurations, level of seismic design and material properties in a population of buildings. All these factors affect the response of buildings. For example, by varying the material properties in a single design of reinforced concrete structure, Rossetto (2006) observed average variation (COV) of 22% in the maximum relative floor displacement of the analysed buildings (used by the author to indicate the damage state of the building). The peaks, frequency content, cycles and duration of earthquake traces recorded from an earthquake of equal magnitude and at similar distances from an earthquake source can also vary significantly. As the structural dynamic response is susceptible to all these parameters, suites of accelerograms must be adopted in the structural analysis. For example uncertainty in ground motion introduced an average coefficient of variation of 27.0% in the ground motion parameter (spectral displacement) adopted by Rossetto (2006), resulted in a 37.2% variation in the structure response parameter (maximum relative floor displacement). However, no formal guidance exists for the selection of accelerogram suites for use in vulnerability curve generation.

A variety of analysis procedures have been used to assess the response of structures under earthquake actions, ranging from the elastic analysis of equivalent single degree of freedom systems (Mosalam *et al.*, 1997), to non-linear time history analyses of 3D models of RC structures (Singhal and Kiremidjian, 1997). Earthquakes apply cyclic loads to buildings which respond dynamically, and often reach highly inelastic levels of response. The inhomogeneous, anisotropic and composite nature of many construction materials complicate the analysis. Simplified analysis procedures may not be able to capture these complexities. Many existing analysis codes also have difficulties converging when structures are subjected to large demands, and numerical collapse may precede structural failure. This is problem is becoming less so with the advancement of powerful finite element codes, computer processors and memory.

Irrespective of the method of analysis used, physical damage must be interpreted from the analytical structural response. A number of damage indices have been proposed for this purpose (summarised in Ghobarah *et al.*, 1999). Some of the more frequently adopted damage indices (Park and Ang, 1985) require the response to be calculated using time-history analyses, which, as previously stated, is not ideal in a vulnerability assessment due to the additional computational effort. Furthermore, the majority of existing damage indices are calibrated with very little experimental data. Where the latter is done, monotonic tests on small-scale structural specimens are used, which do not represent the damage processes in under the dynamic loading of earthquakes. Two exceptions are the empirical equations for reinforced concrete member ultimate and yield rotations proposed by Rossetto (2002) and Panagiotakos and Fardis (1999). In the former, 721 monotonic and cyclic tests were adopted, however, as can be seen from Figure 6 a large scatter of the experimental data is observed around the empirical expression. Considerable uncertainty in the risk evaluation is therefore introduced by the damage index. In terms of a risk assessment over an extended area, such variations in damage classification can lead to much more expensive and possibly unnecessary interventions.

Figure 6: Comparison of predicted and observed ultimate member chord rotations for 721 tests



Source: Rossetto (2002)

Hybrid vulnerability curves are derived when data from the different sources is combined. This is done in an attempt to compensate for the scarcity of observational data, subjectivity of judgemental data and modelling deficiencies in analytical procedures. Singhal and Kiremidjian (1999) adopt a Bayesian technique to update analytical curves for low-rise frames with observational damage data from a tagging survey of only 84 buildings affected by the Northridge earthquake (USA, 1994). Experimental data can also be used to validate and update vulnerability curves. Some testing of large and reasonably realistic structures under earthquake loading has taken place in recent years. However, the large monetary and time costs involved mean that only a very limited number of parameters can be investigated and parametric variations are not possible.

In summary, the development of simple tools for vulnerability assessment has been slower than for the other constitutive elements of seismic risk. This is mainly due to the expense associated with the dynamic testing of structures, and the large computational effort and specialist programmes required for the analytical simulation of building population seismic response. Existing vulnerability relationships only cover a limited number of regions and structural types. They are mostly developed from the effort of individuals rather than a united research community, and little agreement exists regarding the derivation methodology, performance criteria and ground motion characterisation adopted for their development. Furthermore, considerable uncertainties are associated with the curves, which can have a large influence on the final risk assessment. In a study by Grossi *et al.* (1999) of the towns of Long Beach and Oakland in California, taking the 10 and 90 percentile bound curves for the vulnerability relationships lead to a variation between 31% and 300% of the mean homeowner loss predicted.

Exposure

The link between structural damage and economic loss or human loss is made via knowledge of the exposure. Depending on the purposes of the risk assessment, exposure can mean the type and number of buildings, their location, value, use and occupancy. Data on inventory also feeds directly into the vulnerability evaluation. However, obtaining exposure data to the desired level of refinement is extremely difficult. Few inventories of cities exist. Those existing have not been carried out for the purposes of an earthquake risk assessment and hence may not report the details required. For example reinsurance companies commonly used tax assessors' data and insurance company data which will detail the location of buildings, their height, value and sometimes the structural type but may not cover all buildings in a location.

Approximate methods exist for the estimation of economic losses from building damage, especially when urban areas (rather than single buildings) are analysed. Typically a relationship between damage states and percentage of building replacement costs are adopted, however these are commonly based solely on judgement rather than market research. The indirect costs (due to building or industry closure) are more difficult to evaluate and can only be based on experience from previous earthquake events. Only very crude methods exist for relating building damage to human losses. Occupancy data is typically available in the form of a population density figure, and only judgement is used to assign a value of deaths and injuries to structural damage.

Risk in Design

The objective of earthquake engineering is to provide an adequate level of seismic resistance in engineering works, through the reduction of vulnerability, at an acceptable cost. The acceptable level of risk is therefore determined by a balance between the cost of providing earthquake resistance and the losses that this investment will prevent. Again it is to be emphasised that losses may mean monetary loss or life loss or most commonly both.

Probability-based limit-state design is the basis of most new structural design standards and specifications worldwide (Ellingwood, 2001). Conventional methods of seismic design aim to provide for life-safety (through appropriate strength and ductility) and damage control (through appropriate serviceability drift checks) (Ghobarah, 2001). Reliability theory is used in order to account for uncertainties in loading conditions (generated by humans, wind, snow, etc.), and resistance (variation in material quality, workmanship, etc.). The First Order Reliability Method (FORM) has been used in most recent codes to set partial factors to material parameters and weighting factors for load combinations, in order to account for their variation. For structures under earthquake loads, however, the responses often become dynamic and non-linear and have different hysteretic behaviours due to yielding and brittle fractures of components. The problem is not generally amenable to FORM since it is difficult to determine the limit state function under these circumstances (Wen, 2001). It is also not correct to be conservative in the case of seismic design where you are designing for a certain failure mechanism and conservatism can either change the failure mechanism or result in grossly oversized members and increased expense.

In most seismic codes the incorporation of uncertainty has been limited to the selection of design loads based on return periods (Wen, 2001). This load is then used with a series of factors to represent the effect of structural period, loading characteristics, site conditions, structural inelastic response, structure importance etc. These factors are largely determined by judgement and are often calibrated so that the resultant designs do not deviate significantly from the acceptable practice at the time (Wen, 2001). The design earthquake in most existing seismic codes is considered to be that associated with a 10% probability of being exceeded in 50 years (the expected life span of a modern building). This corresponds to an earthquake with a return period of 475 years. The earthquake size associated with this return period changes with the seismicity of the area in which the buildings are being constructed/assessed. However the reason why this specific level of exceedence probability has been chosen for building designs worldwide is not known. The origins of this value are obscure and are being criticised by the seismic research community. Irrespective of the return period earthquake for design, building codes for countries where infrequent natural hazards occur or where there is an incomplete historical record of past natural disasters, often inadequately account for the seismic hazard. Their hazard or zoning maps do not adequately represent the frequency of occurrence or potential magnitude of earthquakes, (Rossetto, 2007). Due to the large influence of seismic hazard uncertainty on risk this results in the assumed design risk not being that intended.

In order to set the design criteria the level of risk that is socially acceptable must be identified. Socially acceptable risk is the probability of failure (damage) of infrastructure that is acceptable to governments and the general population in view of the frequency and size of natural hazards, and the infrastructure use, importance and potential consequences of its damage. For example, it is unacceptable that a nuclear power station be damaged by any natural hazard event; the acceptable risk is, therefore, zero. In most cases constructing buildings and infrastructure that can fully resist the largest earthquake is uneconomical (and often unjustified due to the rare nature of some natural hazards). Hence a limited risk is accepted (Rossetto, 2007). Determining what is an acceptable risk involves the use of an acceptable decision process. Paté-Cornell (2002) list the elements of an acceptable decision process in their paper, which amongst others include a sound legal basis with clear understanding of individual and societal risks and treatment of economic effects, a communication system, a public review process, a conflict resolution, monitoring and feedback system.

In recent years a new paradigm in seismic design has arisen called 'performance-based design' (SEOAC 1995, ATC 40 and FEMA 273). This involves the association of desired performance objectives (e.g., operation and severe damage but life-safety ensured) with different hazard event return periods (e.g., a very rare event and largest possible event) for the determination of the loading for the building design (Rossetto, 2007). This framework allows a clearer incorporation of risk in design and to some degree empowers the building owner to decide what level of risk is acceptable to them.
Performance requirements are qualitative statements, whereas limit states for design are required to be quantitative evaluations of structural response (Ellingwood, 2001). This creates a problem as the traditional design of buildings involves supplying sufficient resistant to the applied actions expressed in the form of forces. However, it is accepted in the field of earthquake engineering that there is little correlation between the applied forces and observed damage. Performance-based design has therefore lead to the development of new methods of design that involve the use of displacements instead of forces in the design of seismically resistant buildings in view of the better correlation of displacements and deformations with observed damage. Many such approaches have been proposed recently for both the design and assessment of buildings (Kowalsky, Priestley and MacRae, 1995; Calvi and Kingsley, 1995; Priestley, 1997, 1998). However there is still live debate as to the particulars of these methods (e.g. the choice of stiffness, Miranda, 2006). Furthermore the tools necessary for their successful implementation are still required. For example, reliable relationships between structural element deformation capacity with their detailing and sizing parameters are needed, as are experimentally calibrated relationships between damage and deformation of structural elements.

However, the derivation of good standards and guidelines for seismic design are insufficient to provide a controlled level of seismic risk in buildings. The correct application of the seismic codes requires skilled engineers, architects and builders and effective enforcement and inspection procedures. For example, good seismic codes of practice exist in India, but their non-enforcement, combined with poor inspection procedures, led to the failure and heavy damage of 179 high-rise reinforced concrete buildings in Ahmedabad, 230 kilometres away from the epicentre. Damage to port operations and industry resulted in approximately US$ 5 billion of direct and indirect losses (MAE, 2001). Enforcement procedures have, however, also been found to be ineffective in some developed countries, as was highlighted by the Izmit earthquake (1999) in Turkey. The implementation of seismic codes in a framework of quality control and enforcement is therefore crucial.

Conclusions

It has been shown that the determination of seismic risk involves the integration of exposure, seismic hazard assessments and vulnerability studies. Each element of the risk model is associated with large epistemic and alleatoric uncertainties which are made worse by the lack of large quantities of good quality data, poor modelling, approximate analyses and near impossibility of experimental verification. SSHAC 1997 summarise the current situation when they state: 'No amount of statistical analysis, no matter how rigorously based and carefully done, can totally compensate for the incompleteness of available data and the defects of our evolving scientific knowledge'. However, as earthquake engineering is a relatively new science the situation can be expected to improve. The proliferation of accurate inventory data is expected in the near future, due to the current development of GIS based survey procedures and programmes for the interpretation of aerial photography. Concerted efforts are being made to improve the prediction of seismic hazard. Attenuation models for the prediction of spectral ordinates, which account for the effects of source, site and path on ground motion characteristics, have been developed. Following observation of the poor performance of existing buildings in recent earthquakes, a multitude of tools and procedures have been developed for the vulnerability assessment of single structures and building populations. Risk needs to be better incorporated in seismic design codes. However, new paradigm that will help move the power of decision of acceptable risk from code-makers to building owners is being developed. Despite all these expected improvements, earthquake resilience will only ever be achieved if the importance of code enforcement, quality control and maintenance are appreciated by governments worldwide.

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