Report to the Canterbury Earthquake Royal Commission

STRUCTURAL DESIGN FOR EARTHQUAKE RESISTANCE: PAST, PRESENT AND FUTURE

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Contents

Summary

Natural disasters teach lessons on preventive measures and preparedness to mankind and earthquakes are no exception. All previous earthquakes which have caused structural collapses and fatalities have also helped engineering communities to improve seismic design provisions throughout the world. Amendment of design practices after a major earthquake often tempts the designers to believe that an absolute safe design practice had been achieved; a false sense of confidence which would be shattered by the next big earthquake. In reality, this sequence of "learning from disasters" and "improving the design practice" seems to be never-ending.

In the last century, seismic design has undergone significant advancements. Starting from the initial concept of designing structures to sustain no or minimal damage (i.e. loosely referred as responding elastically) during an earthquake, the modern design philosophy allows structures to respond to seismic ground motions in an inelastic manner, thereby sustaining damage in earthquakes that are significantly less intense than the largest possible ground motion at the site of the structure. This major shift has occurred through several transitional phases such as load and resistance factor design, limit state design, capacity design, performance based design etc. These phases were founded on the new knowledge unearthed by the then ongoing research and novel concepts developed at the time leading to that phase. Current multi-objective seismic design methods are characterized mainly by their aims to ensure life-safety by preventing collapse in large and rare earthquakes and to limit structural damage in frequent and moderate earthquakes. Lately, more emphasis is being given to financial implications of a seismic event rather than on measures of structural response and/or damage. This has led to a concept of loss optimization seismic design, which looks likely to be the basis for future seismic design approaches.

1. Introduction: Earthquakes and Seismic Hazard

Earthquakes are defined as the phenomena of fault rupture which releases the strain energy stored inside the earth's crust. The release of the energy results in vibratory waves propagating through the surface in all directions. While doing so, the earthquakes create several hazards such as: surface rupture, ground and slope failure, tsunamis, ground shaking etc. Although all of these hazards pose threat to infrastructures, what is commonly termed as seismic design considers the ground shaking hazard only. The ground shaking hazard at a site is a combination of hazards due to all possible earthquake sources (e. g. tectonic plate boundaries and faults) in the vicinity of the site. The contribution of each earthquake source to ground shaking at a site depends on the magnitude of earthquakes originating at the source, the source-to-site distance, the directivity of the fault rupture process, and the geological condition of the soil between the source and the site.

When earthquakes strike, functionality of manmade infrastructures like buildings, bridges, dams, roads, canals, pipelines may be disturbed. The extent of disturbance, however, depends on the severity of the earthquake-induced ground shaking at the site and the robustness of the infrastructure. The robustness of an infrastructure depends on the design, materials, and construction practice prevailing in the region at the time when the infrastructure was built. Similarly, the ground motion severity depends, among others, on the soil conditions at the site and on the proximity of the location to tectonic plate boundaries and inter-plate faults. Seismic design aims to avoid/minimize the damage to infrastructures due to ground shaking resulting from all possible earthquake sources in the vicinity. Clearly, planning and constructing earthquake-resistant infrastructure is a multi-disciplinary task which requires a sound knowledge of engineering seismology and structural engineering.

Structures are designed to safely resist a combination of actions; such as self weight (i.e. dead loads), superimposed (i.e. live) loads, snow loads, wind forces and earthquake forces. Where natural hazards such as earthquake, wind, snow do not pose a major threat, structural design is mainly governed by the dead and live loads. Such designs are also known as “gravity design”. On the other hand, where the earthquake induced ground shaking is a major hazard, the design load is dominated by seismic forces and such designs are known as “seismic design”. Seismic design is significantly different from gravity design as seismic loading on structures is highly uncertain and can occur very infrequently. Thus, it is uneconomical to design structures to sustain the maximum likely ground motion an earthquake rupture can produce, and it is, therefore, a common practice to design structures to respond inelastically to earthquake shaking, but allow sufficient ductility to prevent structural collapse. The method to design structures to resist earthquake induced forces (commonly called “seismic design”) has undergone major advancement in the last few decades. This chapter summarizes the progress of seismic design philosophy from the past to the present and also projects the future of seismic design as indicated by the current research trend. While doing so, the main emphasis is given to buildings but the discussion is not facility specific; the historical advances of seismic design philosophy described herein are equally relevant to other infrastructures as well.

2. Evolution of Structural Design Concepts

During the initial phase of evolution of design concepts, “structural design” involved estimating the structural size so that it could withstand a perceived level of maximum expected load. When structures started to be designed, no consideration was given to any other aspect apart from load and resistance. Notwithstanding the regular amendments, all structural design philosophies, in general, are governed by the “capacity greater than demand” criterion which is commonly expressed mathematically as:

$$S_d \le \phi S_n \tag{1}$$

where, S_n is the nominal capacity of the structure and S_d is the required demand. The demand corresponds to design actions applied to the structure. In order to account for the uncertainty in estimating the capacity of a structure, a factor ϕ (less than one) is commonly used to multiply the nominal strength estimated from an analysis. Instead of using this strength reduction factor, material factors are also used to modify characteristic strengths of materials (e. g. the cylinder strength of concrete and measured yield stress of reinforcing bars) and the demand is compared to the capacity calculated based on the reduced material strengths. In some design approaches, more than one factor are employed to ensure that the *estimated minimum capacity* is greater than the *perceived maximum demand*. Demand is commonly expressed either in terms of design load (the term “load” usually refers to gravity, earthquake and wind induced demands are expressed as seismic/wind “forces”) or corresponding stress in the critical part of the structure, and capacity is measured in terms of structural resistance (maximum load that could be resisted) or the strength of the materials used. Since its inception, the underlying principle of structural design has always been “capacity greater than demand”, which has been interpreted differently in the different structural design concepts that have evolved throughout the last century.

2.1. Working stress design method

The concept of working stress design method (also known as allowable stress design ASD) started around the beginning of the 20th century. In this method, structures or members are proportioned such that the stresses induced due to prescribed working loads are less than the allowable stresses (representing the elastic limit) specified in the codes. In other words, the service load should not exceed the allowable load, which is calculated as the nominal strength divided by a factor of safety to account for uncertainties. Designed structures are intended to remain within elastic range and linear analysis is sufficient to estimate the working stresses. All uncertainties (in demand and capacity) are combined in a single factor of safety which is used to reduce the ultimate strengths of materials to be used as the allowable stresses.

2.2. Ultimate strength design method

The concept of ultimate strength design started to evolve in 1950s and this design concept started to appear in the design codes from the late 1960s. Ultimate strength design is based on the requirement that the design load effects multiplied by the specific load factors are less than the computed nominal strengths multiplied by specified strength reduction factors. As explained earlier, the strength reduction factor is not needed if the nominal strength is calculated using the nominal material strengths divided by appropriate material factors. The concrete design codes were the first to adopt this design philosophy. Steel design codes adopted the ultimate strength design in the form of Load and Resistance Factor Design (LRFD) but also allowed the working stress method to be used. Timber is the only material that appears to be still following the working stress design. This is because timber is basically a brittle material, and ultimate strength and elastic strength are essentially the same. One of the major advantages the ultimate strength design (or LRFD) offers over the allowable stress design is the use of separate factors to account for the uncertainty in capacity and demand. The factors to multiply the capacity (i.e. strength reduction factors) are less than one and differ for different materials and mechanisms; i.e. smaller values are used when there is less confidence on the estimation of the capacity corresponding to a type of failure mode (shear, flexure etc.). As the variation in concrete strength is more than in steel, typically a smaller factor is used for concrete than for reinforcing and structural steel. Similarly, the factors to multiply the demand (i.e. the load factors) are greater than one and different factors are used to multiply different forms of loads (i.e. live, dead, wind, seismic etc), which are then combined to come up with the factored design load. In determining the specific magnitude of the load factors, more deterministic loads are given lower factors than highly variable loads. For example, as live loads are more difficult to predict than the dead loads, typical live load values are multiplied with a greater load factor than that used for the dead load.

2.3. Limit state design method

Limit state design is an extension of the concept of LRFD, the only difference being that it requires the structure to satisfy more than one design requirement (termed as limit states). A limit state is a set of performance criteria (e.g. vibration, crack width, deflection, buckling, and collapse) which must be met when the structure is subjected to a level of load. In general, two principle limit states are used: the serviceability limit state (SLS) and the ultimate limit state (ULS); although an intermediate limit state (i.e. damageability limit state) is also used sometimes. SLS is intended to ensure that the structure remains functional and no discomfort is caused to the occupants through excessive sway/deflection/vibration when subjected to routine loading. A structure is considered to have satisfied the SLS criteria if the estimated deflection, vibration, crack widths are within permissible limits specified in the codes. Elastic methods of analysis are generally acceptable for checking SLS criteria. A structure not fulfilling the SLS criteria will not necessarily fail structurally. ULS is to ensure that a designed structure does not collapse when subjected to the peak design action. A structure is deemed to satisfy the ULS criteria if all the design strengths (nominal strengths in flexure, shear etc multiplied by the corresponding strength reduction factor, or nominal strength calculated by using factored material strengths) equal or exceed the design actions (sum of load factored actions).

3. Evolution of Seismic Design

The concept of seismic design started in early 20th century. Discussions on deficiencies of structural systems and the resulting damage due to the 1906 San Francisco earthquake can be found in abundance in the literature. Since those days, people in seismically active countries like USA (especially the west coast), New Zealand, and Japan have been working towards forming a robust earthquake resistant design. The first active step in mitigating seismic risk was taken by the Seismological Society of America in 1910, when it identified three earthquake-related issues requiring further investigation: phenomenon of earthquakes (when, where and how they occur), the resulting ground motions, and their effect on structures. The seismic performance of then-existing structural forms had been perceived to be weak. Records show that structural engineering communities throughout the world had understood that earthquakes expose structures to lateral forces that are different from the vertical gravity loads and structures need to be specially designed to withstand earthquake induced ground shaking. A review of historical seismic design codes of different countries reveals that the definition of seismic safety has undergone gradual changes towards making it more concise, specific and performance-based. To accommodate these sophistications, several important concepts have evolved through the years. Through all these revisions of seismic design philosophies, the underlying design concept of "capacity greater than demand" has remained pivotal. Nevertheless, the meaning of the general terms "capacity" and "demand" has been interpreted differently at different stages of this journey.

3.1. Strength based design

Until the 1960s, seismic design provisions were largely based on "induced stress less than allowable stress" criterion. The induced stresses were calculated by applying lateral seismic design forces which were taken as a fraction of the weight of the structure and the structure was designed such that the stresses induced by the design seismic forces when combined with gravity loads were less than the allowable stress levels. This was the "working stress method" applied in seismic design. In seismic design a truly elastic design approach is difficult to correlate with expected structural response. After all, by definition, a design earthquake is an ultimate-strength event. From the 1970s onwards, the concept of "ultimate strength design" started to appear in the seismic design codes. This change also brought the need to take inelastic behavior into account; mainly to conduct nonlinear analysis to calculate the ultimate strength of a member. The ultimate strength based seismic design basically involved calculating the design strengths and comparing them against factored seismic design actions.

3.2. Multi-objective prescriptive design

When the ultimate strength design method was being commonly used in seismic design, earthquake engineers realized that just ensuring that a designed building does not fail in an ULS earthquake is not enough and the building also needs to respond to smaller and more frequent earthquakes without causing any significant discomfort to its occupants. This led to the use of limit state design where both the serviceability and ultimate limit states would need to be satisfied. The serviceability criteria required buildings to sustain no or minimum damage (loosely referred to as remaining elastic) in frequent earthquakes (typically with 50% probability of exceedance in 50 years) and the ULS required the building not to collapse (to ensure life safety) in a design level earthquake (5% probability of exceedance in 50 years). This was a significant advancement as for the first time a building needed to satisfy more than one performance criteria. This marked also the beginning of multi-objective performance based seismic design, where multiple performance criteria corresponding to different levels of earthquakes (usually specified in terms of their probability of occurrence) are checked in a precise and quantitative manner. An example of this prescriptive approach can be found in the Uniform Building Code (UBC) which specified the performance requirements for a building as shown in Table 1.

Table 1: Required building performances for different levels of ground motions (UBC)

Earthquake intensity	Frequency of occurrence	Desired performance
Minor	Several times during the building's service life	No damage to structure or non-structural components
Moderate	One or more times during the building's service life	No significant damage to structure and limited damage to non-structural components
Major	Rare event as large as any experienced in the vicinity	No collapse of structure or other damage that would create a life safety hazard

Similarly, Structural Engineers Association of California (SEAOC) seismic design manual stated that the lateral force requirements are to produce structures that should be able to resist: a small earthquake with no damage, medium earthquake with some nonstructural and contents damage but no significant structural damage, and the largest earthquake predicted at the site with significant damage of structural components but without structural collapse. Design of structures following today's design standards, although having many different forms and equations, generally still follow the same philosophy presented in the SEAOC document mentioned above.

One of the features of these guidelines is that the demand and capacity are not concisely defined; vague and subjective terms such as "moderate", "one or more times", "limited damage" are used. Three levels of performance against three different levels of earthquake are required, but only the largest earthquake intensity (i.e. major) is quantified as 10% probability of exceedance in 50 years. The ambiguity of the definitions can lead to wide variations in the interpretation of the code.

3.3. Performance based seismic design

Until late in the 20th century, all design codes had prescriptive guidelines to achieve serviceability and safety. In doing so, the codes specified a common value of response parameter that the designed structures shall not exceed in limit state events. The concept of performance based design evolved when designers started realizing that such a prescriptive design was not always the most appropriate method. Different structures have different performance requirements and it is not appropriate that the same prescriptive criteria be used for designing different structures. For example, the ULS for a water tank refers to cracking as no cracks should be permitted to enable the tank to store water which is its main purpose, whereas ultimate state for a residential building is prevention of collapse (to ensure life safety). Obviously, these two limit states correspond to drastically different values of critical response parameters (such as lateral drift).

In performance based design, the aim is to satisfy the performance requirements of a structure rather than to ensure that the response is within a prescribed limit. The performance requirements are structure specific; for a residential building severe damage in an extreme event is permitted whereas any damage in a hospital or an emergency facility (even in an extreme event) is required to be minor so that the functionality of such important facilities are not interrupted after an earthquake. Currently, many seismic design codes require structures to satisfy more than one seismic performance requirement. In such a multi-level seismic performance based design concept, in addition to verifying the prevention of collapse in an extreme earthquake, structural performances in smaller levels of earthquakes also need to be checked.

Typically, required performances against three different seismic hazard levels are specified in modern performance based seismic design codes for buildings. The three seismic hazards are generally categorized as frequent earthquakes (usually with 100 years return period; 50% probability of exceedance in 50 years), design basis earthquake (DBE) with 475 years return period (i.e. 10% probability of exceedance in 50 years), and maximum considered earthquake (MCE) with 2475 years return period (i.e. 2% probability of exceedance in 50 years). The actual earthquake intensities corresponding to these hazard levels depend on the seismicity of the location of interest. As shown in Figure 1, the required performance of buildings in these three hazard levels depends on the importance of buildings. Obviously, buildings that house emergency facilities are more important than normal residential buildings and need to be functional even after rare earthquakes. In general, performance requirement can be categorized into four classes as operational (functioning fully after an earthquake), immediate occupancy (slightly damaged but any minor repair could be done without disrupting the function of the building), reusability (also referred to as life safety) (damaged but reparable although the building may need to be evacuated for repair), and collapse prevention (does not collapse although the building may be severely damaged requiring demolition).

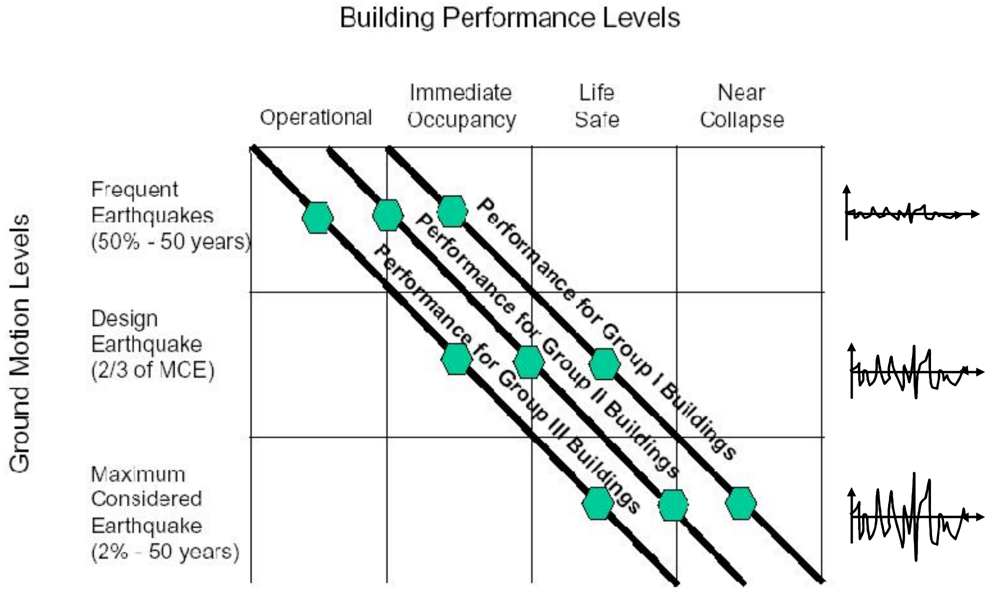


Figure 1: Framework for performance based seismic design of buildings

The first and the last categories can be verified more easily than the remaining two; "operational" means the structure must avoid significant damage and "collapse prevention" means the structure remains standing regardless of the extent of damage when subjected to the specified seismic hazard level or its equivalent action. The interpretation of the remaining two categories can be subjective. For the verification of immediate occupancy slight inelastic response along with minor damage, such as cracking and minor yielding are acceptable, whereas for reusability any reparable damage such as spalling of cover concrete are accepted but irreparable damage such as buckling and fracture of reinforcing bars are not. In extreme earthquakes (i.e. MCE), normal buildings are required to satisfy the collapse prevention criteria whereas more important buildings may be required to satisfy the reusability criteria. From the previously described categorization it appears as if life safety is compromised in MCE for normal buildings, but it is not actually so. In contrast to the names of the categories, threat to life-safety originates mainly from collapse rather than from severe damage; therefore life-safety will be achieved if collapse prevention is ensured. In DBE, normal structures are required not to sustain severe (irreparable) damage (i.e. reusability criteria), whereas more important structures are required to be available for immediate occupancy/use. Similarly after frequent earthquakes, normal structures are allowed to undergo minor damage, the repair of which does not require the building to be closed (i.e. immediate occupancy), whereas more important structures are required to remain perfectly undamaged (i.e. operational).

4. Evolution of Seismic Demand

As is obvious, the safety of a structure designed with "capacity greater than demand" principle depends on how closely the perceived design seismic action represents the upper bound seismic demand and also on the capability of the adopted analysis method to reliably estimate internal stresses and strains due to the design seismic action. Since the inception of seismic design practice, the design seismic force has been expressed as a lateral force coefficient that is multiplied by the total weight to calculate the design lateral seismic forces. The values of this coefficient were decided based on the (limited) knowledge of the historical earthquakes in the region at that time. A review of seismic design codes shows that the seismic design coefficients have undergone gradual increase from 0.015 to 0.4 or even higher at some locations, and these increases have generally been triggered by the occurrence of a larger earthquake making the designers realize that the demand at that location is higher than what was being recommended by the design codes until that stage. Although the prediction of the capacity of structures has more or less remained the same, it is the demand which has been revised regularly. When a structure designed to remain elastic using a smaller design seismic force is exposed to a bigger earthquake, the imposed seismic demand exceeds the structural capacity. As no intentional ductility was provided in the structure with the expectation that it will remain elastic, it could collapse in such bigger earthquakes. The designers would then realize the need to retrofit or strengthen all structures built earlier for a smaller level of demand to bring their capacity to a level in excess of the current (increased) demand. Nonetheless, with another bigger earthquake down the track, the designers would be embarrassed to know that the seismic demand was still grossly underestimated to make the retrofitted structures vulnerable to collapse.

4.1. Early age: Common seismic demand

In the early versions of seismic design codes, the demand expressed in terms of a fraction of the structural weight was constant for all structures, and no importance was given to the type of structure and type of soil underneath it. Within the territorial scope of applicability of a code, no consideration was given to the different levels of seismic hazard at different locations. For example, the 1955 seismic loading code of New Zealand specified that lateral seismic design actions were found by applying lateral forces equal to 0.08 of the weight at each floor level or by using a lateral force coefficient of 0.12 at the uppermost level and a value of zero at ground level with linear interpolation between these points. The same lateral force coefficients were used throughout the country.

4.2. Middle age: Elastic demand spectrum (focus on strength)

In the next stage, the variation in the seismic hazard of different locations started to be accounted for by dividing a country/region into different seismic zones and by assigning different seismic hazard factors for each zone. This was an important advancement which would lead to seismic microzonation of different seismic regions based on probabilistic seismic hazard analysis. In the next stage, the designers realized that the seismic demand not only depends on the location of the structure, but also on the structural dynamic parameters, mainly the fundamental period of the structure. In this phase, an equation of the following nature evolved in seismic design codes worldwide

$$V = C_d(T)ZW \tag{2}$$

where $C_d(T)$ is design base shear coefficient which is a function of the structural period T, Z is the zone factor which represents seismic hazard of the location and is usually presented in terms of the likely peak ground acceleration (PGA) divided by the gravitational acceleration g and W is the weight of the structure. Although not included in the equation, other factors were also used in the earlier versions of seismic design codes to account for different materials, structural types and limit states.

This phase also marked the beginning of the use of elastic design spectrum (Figure 2) to obtain the design seismic force. The elastic design spectrum gives the maximum response acceleration demand of structures with different natural periods. As can be seen in Figure 2, the maximum acceleration of an infinitely rigid structure (i.e. zero period) is equal to the PGA (i.e. a_g). The maximum design acceleration is constant at 2.5 (some codes use 3.0) times the PGA for the constant acceleration range between natural periods T_a (~0.1 s) and T_v (~0.4 s). In the constant velocity range between T_v and T_d (~3.0 s), the maximum velocity is constant and the maximum design acceleration is inversely proportional to the natural period. For a very flexible structure with natural period greater than T_d, the maximum displacement remains more or less constant (equal to the maximum ground displacement), and in this region the maximum design acceleration is inversely proportional to the

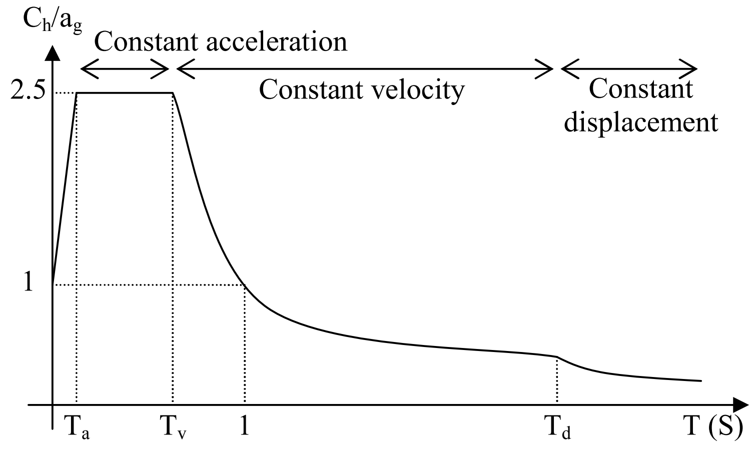


Figure 2: Typical elastic design spectrum

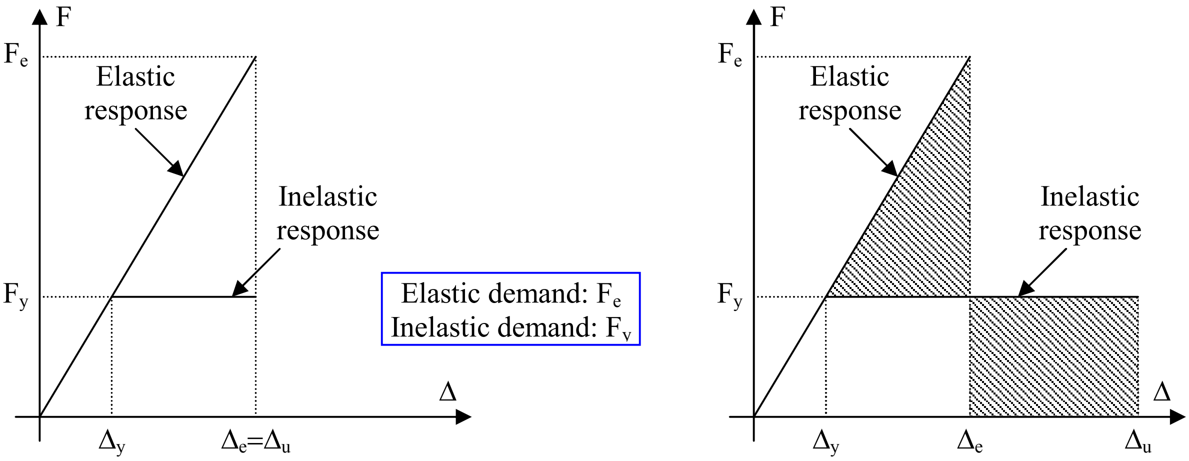
square of the natural period. In many design codes, the constant displacement region is not included as the natural period of common structures is less than T_d.

In the next phase, the effect of soil was taken into account in the design spectrum. Earthquakes originate inside the earth's crust and the ground motion at the site is derived using attenuation relationships which take into account the filtering (of high frequency) and fading (of amplitude) when a wave travels through a horizontal distance. Thus obtained ground motion hence represents the motion inside the earth's crust (not at the ground surface), and the vertical travel of the wave from the original depth to the base of a structure at the ground surface also needs to be taken into account in predicting the ground motion at a given site. In a way the soil responds to the base motion in a similar way to an elastic structure would respond to a ground motion; i.e. the elastic response acceleration of a very stiff structure is the same as the acceleration applied at its base whereas the elastic response acceleration of moderately flexible structures is higher than the ground acceleration. Similarly, if the soil is stiff the ground motion from the depth is transferred to the surface without much alteration. On the other hand, a ground motion gets amplified in moderate and soft soil. To take into account this effect, soils were classified into discrete categories and different elastic design spectra (similar to Figure 2) were provided for the different soil categories. While calculating the design base shear force using Equation 2, the base shear coefficient $C_d(T)$ was obtained from the spectrum for the relevant soil category.

4.3. Modern age: Inelastic demand spectrum (focus on ductility)

As described earlier, the base shear coefficient obtained from an elastic design spectrum is multiplied with the zone factor and the structural weight to calculate the design lateral seismic force. However, such forces would be significantly large when PGA of very rare earthquakes in earthquake prone regions is taken into consideration for ULS. As such earthquakes may or may not occur in the lifetime of a structure, it will be a waste of resources to design all structures to meet such a high level of demand. This triggered the concept of inelastic seismic design. In modern seismic design practices, the seismic design force is taken to be less than the elastic demand. It means that a structure will remain essentially elastic in a SLS earthquake and respond plastically (i.e. yield) to a maximum considered ULS earthquake. To ensure safety of the structure in a limit state event, it is designed to deform up to a desired level of inelastic displacement without losing its strength.

The ability of structures to deform beyond the elastic limit without losing strength is quantified in terms of displacement ductility, which is defined as the ratio of ultimate displacement, Δ_u, to the displacement at first yield, Δ_y. The inelastic seismic design force is a function of the ductility of the structure, and any reduction in the design seismic force must be compensated by increasing the duc-



Equal displacement principle; $F_y=F_e/\mu$

Equal energy principle; $F_y=F_e/\sqrt{(2\mu-1)}$

Figure 3: Derivation of Inelastic demand

tility demand. Hence, unlike the elastic demand which is fully described by the force F_e, the inelastic demand is described by a yielding force F_y and a ductility μ. As shown in Figure 3, two different approaches are used to correlate the inelastic force demand and the displacement ductility. Equal displacement principle states that the ultimate displacement of an inelastic system is equal to the ultimate displacement of an elastic system with the same initial stiffness. According to the equal displacement principle, the ratio between the elastic and inelastic demand (also called force reduction factor) is equal to the displacement ductility. This is arguably valid for flexible structures with longer period. As an extreme example, for infinitely flexible structures the maximum displacement is always equal to the peak ground displacement regardless of the level of inelasticity. On the other hand, equal energy principle states that the energy dissipated by an elastic system and its inelastic counterpart is equal; thereby rendering the force reduction factor equal to $\sqrt{(2\mu-1)}$, where μ is the ductility of the inelastic system. To account for the reduced seismic demand of the inelastic structure, the equation to calculate the seismic design coefficient $C_d(T)$ was then revised as:

$$C_d(T) = C_h(T)S_p/K_\mu \tag{3}$$

where C_h is the seismic hazard coefficient, S_p is the structural performance factor (used in some countries; e.g. New Zealand) and K_μ is the ductility factor. Following this principle, most modern seismic design codes specify that the force reduction factor or the ductility factor K_μ is equal to the ductility for longer period (taken as longer than 0.7 s) and is a function of ductility (to represent the transition from equal displacement to the equal energy principle) for structures with shorter period.

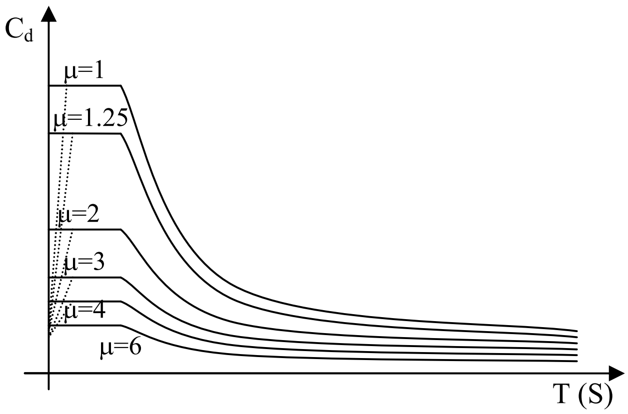


Figure 4: Typical inelastic design response spectra

In modern seismic design codes, the seismic design coefficients (C_d) are normally presented via inelastic design spectra (as illustrated in Figure 4) where spectra are provided for different ductility levels. For flexible structures with long periods the inelastic design force is inversely proportional to the ductility which is in line with the equal displacement principle. Nevertheless, most design codes carry this relationship for short period as well, which is not strictly correct. For short period (below 0.35 s), equal energy principle should be applied and the reduction of design force should ideally be less pronounced than the increase in ductility. Using these inelastic spectra, seismic demand pair of design yield force and ductility can be obtained.

5. Evolution of Seismic Capacity

5.1. Definition and calculation of seismic capacity

With the change in the design approach and the change in the definition of seismic demand, the definition of capacity and the analysis method commonly used to estimate capacity also had to change. As a consequence, the structural seismic performance measures that needed to be calculated and compared to accomplish the design changed. Previously when allowable stress design was used calculation of the allowable stress (which is less than the elastic limit) would suffice for seismic design. Using a fraction of the material strength as the allowable stress was acceptable and no analysis as such was required to calculate the allowable stress (i.e. capacity). In the days of ultimate strength method, the strength of the structure was the "capacity". As structures were designed to remain elastic (in some codes, inelastic response of structures was mentioned but the quantification of inelastic deformation was not required until 1970s), estimation of structural capacity based on material properties using linear analysis or equivalent simplified elastic methods was still appropriate.

When inelastic demand started to be used in seismic design, the capacity had to be represented by a combination of the yield strength and maximum displacement ductility. In order to assess these parameters, nonlinear analysis such as pushover analysis was required. With the use of the limit state method, the definition of capacity started to become more complex as this method required more than one parameter to be assessed. For example, for SLS, limiting values of crack width, deflection, vibration etc were used as the capacity whereas for the ULS, limiting values of deformation, strain, or damage were to be used as the capacity. In the performance based design phase, the definition of seismic capacity became highly subjective and tool-dependent without specific quantitative interpretation of terms such as "immediate occupancy", "life safety", and "collapse prevention". One interpretation of the damage milestones corresponding to these performance categories could be yielding, buckling, and global instability (due to P-delta effects) respectively. Nevertheless, as the method required structures to satisfy performance spanning the whole range of inelastic response, nonlinear analysis with reliable models including material and geometrical nonlinearity was required for the calculation of seismic capacity.

5.2. Seismic performance assessment

Seismic performance is a structure's ability to perform its due functions, such as its safety and serviceability, at and after a particular earthquake exposure. A structure may be considered serviceable if it is able to fulfill its operational functions for which it was designed and is normally considered safe if it does not endanger the lives and wellbeing of those in or around it by partially or completely collapsing. Basic concepts of earthquake engineering implemented in the major building design codes assume that a building should survive the MCE without complete destruction.

Two possible approaches (experimental and analytical) can be used to assess whether a structure satisfies the aforementioned seismic performance requirements. The experimental approach consists of fabricating a scaled model of the structure and subjecting it to a loading which resembles (in effect) the earthquake ground motion, for which the seismic performance is being assessed. With regards to loading protocol, the experimental seismic performance assessment can be divided into three broad categories. The first type is shaking table test where the actual ground motion acceleration time history is applied at the base of the physical model mounted on a shaking table. The next category is called pseudo-dynamic test where the structural model is subjected to a displacement history induced by an earthquake record, which is calculated and fed into the experiment by a simultaneously running numerical analysis using the experimentally measured real-time structural properties. In the third category of tests called the quasi-static tests, the structural model is subjected to a regulated static displacement history (monotonic or cyclic) without any consideration of the actual loading profile in an earthquake, and the results are interpreted to deduce likely response of the structure in real earthquakes.

As conducting experiments every time when the required performances of a design option is to be verified is too resource-demanding, analytical methods of seismic performance assessment is a more viable option. In this approach, analytical model of the structure is created and computations based on principles of mechanics and dynamics are performed to estimate the likely structural response to earthquakes. The range of analyses to be performed depends on the response parameters to be calculated. As mentioned earlier, simple hand calculations based on linear material properties may suffice if elastic stress and strength limits are to be assessed, and a full-fledged nonlinear finite element analysis with appropriate constitutive relationships is required for assessment of performance corresponding to the ULS or multiple performance requirements of modern performance based design.

Several methods can be used for analytical seismic performance assessment of structures. These methods can be broadly classified into four categories: linear static, nonlinear static, linear dynamics and nonlinear dynamic. Calculations based on linear static methods are the simplest to follow and are appropriate for approximating induced stresses in the allowable stress design method, for verifying SLS in limit state design method, or for verifying the immediate occupancy criteria in multi-objective performance based seismic design. Nonlinear static methods of analysis, also commonly known as pushover analysis, are simple yet reasonably accurate in estimating response of some structures, including that in the inelastic range. As material and geometrical nonlinearities are given due consideration in this category of analysis, ULS performance measures such as ductility and structural collapse, which are used to assess life safety and collapse prevention criteria in modern performance based seismic design codes, can also be predicted.

Nevertheless, there are some inherent problems with pushover analysis. Firstly, it has no theoretical foundation to justify its relevance to seismic response of structures; i.e. it is merely for convenience that designers conduct such simplistic monotonic analysis to assess structural response to highly complex loading like earthquakes. Furthermore, pushover analysis is static by definition and in general cannot account for dynamic phenomena such as higher mode effects (although some methods are available to compute these separately and include with combination rules). The effect of higher modes is also a function of the frequency content of the ground motions; in particular the forces in different storeys or components can be affected by higher modes even in structures of moderate height and thus pushover analysis can lead to very misleading results. In this context, dynamic analyses which take into account the effect of higher order modes are more representative of the dynamic response of structures. Nevertheless, linear dynamic analysis, such as modal superposition method, assumes linear behavior for each modal response and is appropriate only for evaluating elastic dynamic properties and not for verifying ULS criteria related to life safety and collapse prevention. Nonlinear dynamic analyses are the most advanced form of analyses which take into account the dynamic response (i.e. higher order modes) of a structure and also the nonlinearity (both geometrical and material) in the system. An example of this category of analysis is non-linear time history analysis.

5.3. Advancement in analytical techniques

Since the evolution of modern earthquake engineering, significant advances have been made in analysis methods due mainly to: (i) an increase in computer processing speeds, thereby enabling non-linear analysis of multi-degree of freedom systems to be performed on PCs; (ii) significant advancement in the development of constitutive models for various aspects of structural behaviors influencing seismic response of structures; and (iii) development of simplified methods which can (relatively) accurately capture the salient features of the detailed analyses. The improvement in structural analysis processing speeds is now at a point where the key effort in conducting structural analyses is the initial development of the structural model, while the run-time for the analyses is relatively short. When repeated analyses are required to investigate structural response to more than one earthquake record, appropriate batch files can be used to automate the process.

The further development of constitutive models has come primarily from an increased number of sophisticated and large scale-benchmark experimental tests on super-assemblages (and at times entire structures). Many constitutive models now have the capabilities to model phenomena occurring over the full range of seismic intensity, from initial behavior of concrete structures before cracking, to yielding, cover spalling, longitudinal bar buckling, and finally global collapse of the entire structure. Also the parallel development of detailed finite element models at the micro-level (material stress-strain formulation) are being used to develop and calibrate macro-level (moment-curvature or force-displacement) constitutive relationships, which are typically used in the analysis of entire structural systems. Sophisticated finite element analysis packages have also become more readily available for both structural and geotechnical systems, thus allowing complete modeling of the complex soil-structure interaction phenomena, which in many instances is critical to the seismic response of a soil-foundation-structure system.

Despite the progress in computational ability, non-linear time-history analyses are yet to be widely adopted in seismic design except for special cases, and it is likely to remain so in the near future. Not surprisingly, a large emphasis has been placed on nonlinear static procedures such as “pushover” analysis for seismic performance assessment. A schematic illustration of how seismic performances of a building are interpreted using a typical pushover analysis result is shown in Figure 5.

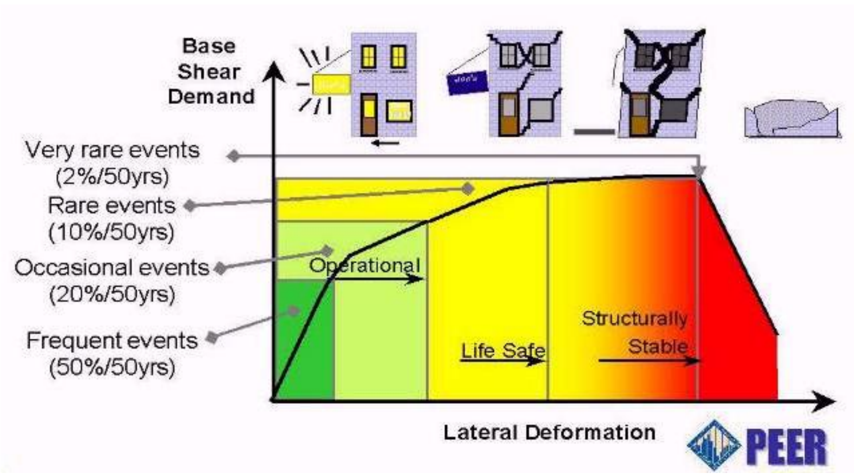


Figure 5: Seismic performance assessment based on pushover analysis

5.4. Capacity spectrum method (CSM)

To make explicit comparison between the structural capacity and earthquake demand, innovative methods such as capacity spectrum method (CSM) has been developed. The CSM is a performance-based seismic analysis technique, which compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of response spectra). The graphical

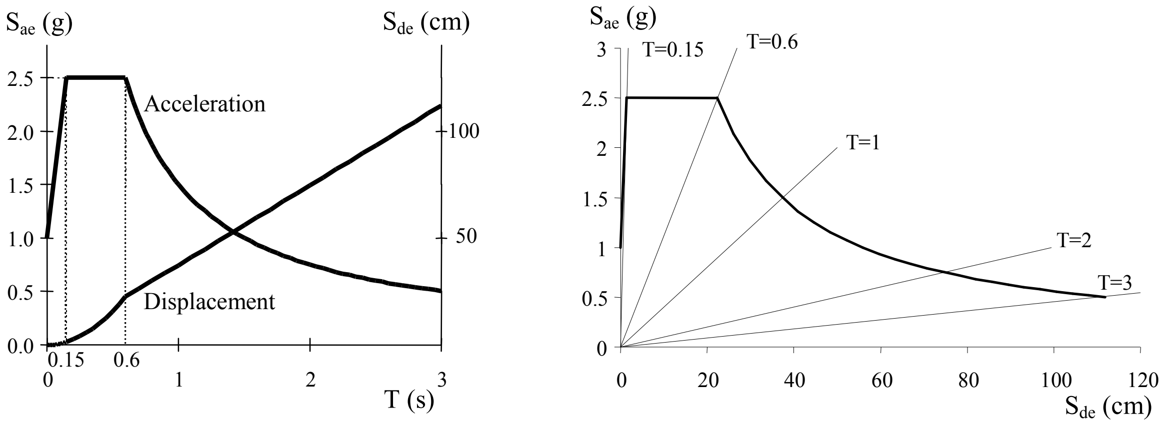


Figure 6: Transformation of traditional response spectrum to ADRS format

intersection of the two curves is called the "performance point", which approximates the likely response of the structure in an earthquake represented by the demand. The CSM is a tool for estimating and visualizing the likely behavior of the structure under a given earthquake in a simple graphical manner. To make the graphical and intuitive nature of the CSM easy to follow, demand spectra are presented in spectral acceleration versus spectral displacement domain, commonly known as acceleration-displacement response-spectrum (ADRS), rather than the traditional spectral acceleration versus period format. The transformation of response spectra from the traditional acceleration-time domain to acceleration-displacement domain is illustrated in Figure 6.

A non-linear pushover analysis of a building normally yields relationship between the shear force at the base of a building and its roof displacement. By converting the base shear force to spectral accelerations and the roof displacements to equivalent spectral displacements, a capacity spectrum in acceleration versus displacement domain can be obtained. Such a capacity spectrum can be plotted together with the response spectrum in ADRS format, and the performance point where this capacity spectrum intersects the response spectrum gives an estimate of the spectral acceleration, displacement, and damage the structure may incur in an earthquake corresponding to the ADRS spectrum being considered. In order to account for inelastic behavior (i.e. yielding) of the structural system, elastic ADRS response spectra generated by applying larger effective viscous damping values are used to represent equivalent inelastic response spectra. To explain the method, Figure 7 shows the capacity spectrum of an elasto-plastically responding structure together with the corresponding inelastic demand spectrum. The point where the capacity curve and demand spectra meet is the required "performance" point and the structure's capacity curve must cross the corresponding inelastic demand spectrum to satisfy the collapse prevention requirement at that level of earthquake. If a structure is required to remain elastic in an earthquake, then the capacity spectrum should cross the elastic response spectrum for that earthquake in its elastic range (i.e. before the yielding point).

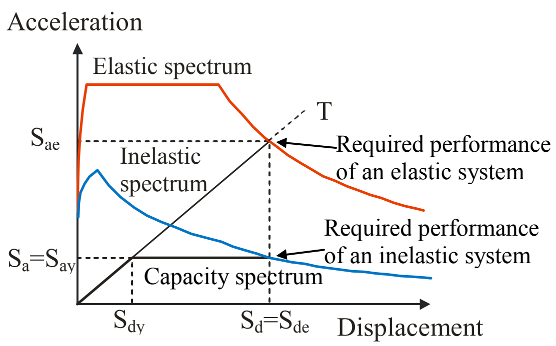


Figure 7: Seismic performance assessment by CSM

Despite its elegance and simplicity, the CSM has been criticized mainly for two reasons: the secant stiffness from the origin to the maximum displacement is a poor representation of the stiffness of the equivalent linear system; and the specified values of equivalent damping appear to provide inconsistent representation of the inelastic response spectra. This has recently attracted more attention and efforts are being made to improve the consistency of its displacement prediction.

6. Seismic Design Approaches in Common Use

6.1. Capacity design

The concept of capacity design started to evolve in 1970s but it was in the 1980s that this started to be implemented in the seismic design codes to an appreciable level. In conceptualizing capacity design, a building was envisaged as a chain and the different components (such as columns, beams, joints, walls) as its links. Based on the underlying principle of "a chain is as strong as its weakest link", the overall strength of the building was correlated to the strength of its weakest component. This analogy can be used not only for a building but for all structural systems. If a reinforced concrete bridge is idealized as a chain then the piers, deck, and the knee-joints are the links; similarly if the chain represents a suspension/cable-stayed bridge, then the towers, hangers, the main cable, and the anchorage are the links. In fact, the same analogy can also be applied to each structural element. For example, if a beam is envisaged as a chain, then its different modes of failure (shear, flexure, bond etc) can be represented as the links. Obviously, the strength of the beam is the minimum of its resistance against these failure mechanisms. Enhancing a beam's strength against the non-critical failure modes is a waste of resource as this would not enhance the overall performance of the beam. In the chain analogy, strengthening links other than the weakest link will not enhance the performance of the chain.

In the context of elastic seismic design, if a structure is idealized as a chain and its components as the links, then the seismic design force can be idealized as two persons pulling the chain at its two ends. The objective of seismic design will then be to ensure that the chain (or its weakest link) does not break when it is pulled with the design seismic force. In order to ensure this, the designers need to (i) identify the weakest link; (ii) accurately (and conservatively) evaluate the strength of the weakest link; and, most importantly, (iii) know with reasonable certainty the higher-bound value of the design seismic force with which the chain will be pulled. The first two are dependent on the structural form and can be achieved if a reliable structural analysis tool is used. As uncertainties are inevitable in this process, one must be careful in interpreting the outcome of the analysis. It must be emphasized here that strength of all links in the chain must be evaluated unless it is blatantly obvious that a link is stronger than at least one other link. Otherwise, there is a chance to unknowingly miss the weakest link; thereby resulting in an overestimation of the overall strength of the chain, which can have disastrous consequences. On the other hand, the third requirement of estimating the pulling force (i.e. design seismic force) is based on the historical data which may not necessarily have fully covered the range of potential threats. Obviously, the pulling force likely to act on the chain (i.e. earthquakes) is a natural phenomenon which is not in complete control of the designers. Aiming to neutralize the effects of randomness of the earthquakes and uncertainly associated with the seismic hazard models used in the design process, the existing prescriptive design process uses many factors. Typically, nominal strengths are multiplied by coefficients less than one (i.e. strength reduction factors) and the design forces are multiplied with factors greater than one (i.e. load factors) to attain a reasonable factor of safety, i.e. difference between the perceived minimum strength of the weakest link and the perceived maximum pulling force.

Structural seismic design adopted in the past did not succeed despite the designers being able to reasonably predict the behavior (including the capacity) of the structure. The main reason for this failure was the underestimation of the demand. Seismology is an ever evolving discipline and the estimation of seismic demand at a time is as good as the seismologists' understanding of the earthquake phenomenon at that time. As shown in Figure 8, structures would generally be designed such that the capacity is reasonably larger than the perceived seismic demand and if an earthquake with seismic forces larger than the demand used in the design occurred, the designed capacity would not be enough to keep the response within elastic limit as intended in the design and the structure might undergo a brittle failure. It is not that designers can accurately predict the seismic demand now; but the current design philosophy is such that the underestimation of the seismic demand does not lead to catastrophic brittle failures. This is done through the principles of capacity design.

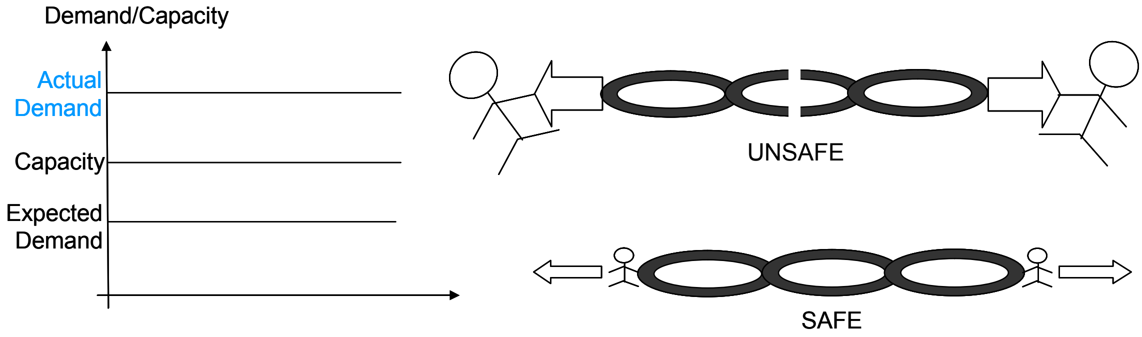


Figure 8: Inherent problem of elastic design

The main motivation behind the development of capacity design was the realization by the designers that they were unable to accurately predict the nature and its forces (in the form of earthquakes), which have repeatedly exceeded the designer's conservative estimates and caused irreparable damage to structures. On the other hand, structures are artificial and are conceived, designed and constructed by people. Hence, with a proper design strategy the structures can be dictated. In other words, designers can force the structures to behave the way they want them to in large earthquakes. To achieve this, two major changes were brought to the strength based elastic design adopted in the past. They are: (i) the elastic design was done away with; and (ii) structures were designed to be ductile (avoid brittle failure when the elastic capacity is exceeded).

In capacity design, the strength (i.e. capacity) of the designed structures is less than the elastic seismic demand. This means that the structures are allowed to deform in a plastic manner. Nevertheless, the structures are designed such that plastic deformations occur only in pre-determined well-detailed locations. This is where the designers needed to dictate the structure. In order to achieve this, favorable plastic mechanisms and favorable locations for these mechanisms must be pre-determined. To ensure that *only* the selected plastic mechanism occurs *only* at the selected location(s) when the structure undergoes inelastic response, these locations and mechanisms are intentionally designed to have less strength than the rest. In chain analogy, one of the links is intentionally made weaker than the others. Now, when the chain is pulled with an increasing force, this weak link will invariably be the first one to reach its strength; this completely rules out the possibility of failure of other components in the structure. Then to avoid the collapse of the structure as a whole, the pre-determined weak link is provided with adequate deformability so that when stretched it elongates but does not break. This is pictorially illustrated in Figure 9.

It is obvious from the above discussion that in capacity design, it is extremely important to ensure that the pre-determined failure mechanism at the chosen location represents the weakest link in the chain. While examining this, the nominal strength of all other failure mechanisms is compared against the upper-bound strength of the chosen failure mechanism. The upper-bound value of strength (also called overstrength) is calculated using upper characteristic material strengths and an appropriate factor to account for the strain hardening of reinforcement at high strain levels. Only when the overstrength of the predetermined failure mechanism at the chosen location is less than the nominal strength of all other mechanisms, it is assured as the weakest link. What will be left after that is only to ensure sufficient deformability in this weakest link.

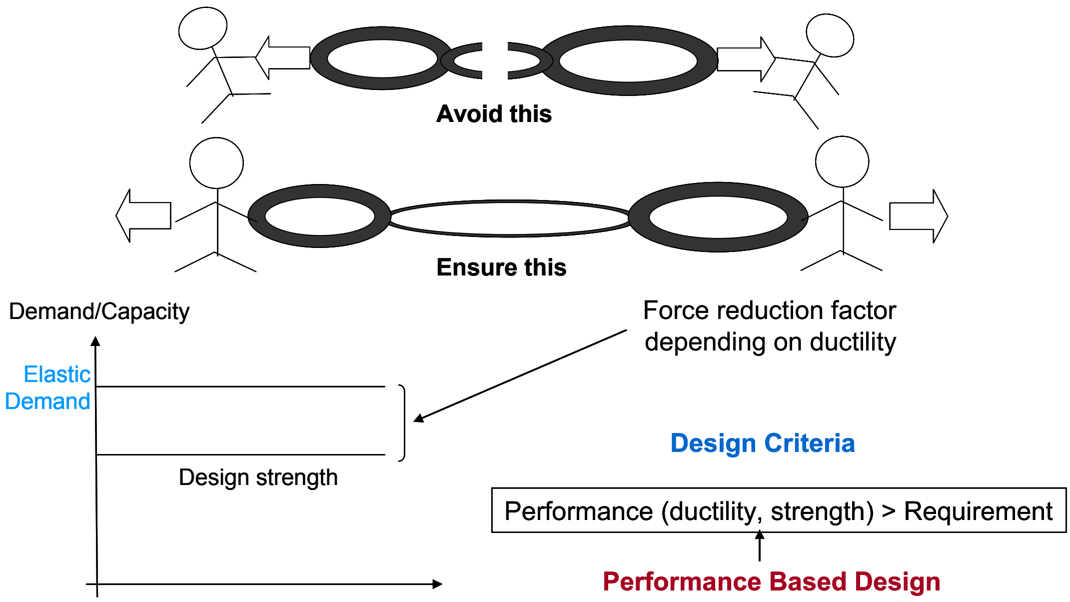


Figure 9: Principles of capacity design

As steel sections are ductile by nature, not much difficulty arises in applying capacity design principles to steel structures and buckling of members and strength of connections are among the key issues requiring special attention in capacity design of steel structures. In reinforced concrete structures, the required ductility is achieved through proper detailing of reinforcement in the critical weak locations (i.e. plastic hinges) which is arguably the most important step in capacity design of RC structures. In addition to providing adequate shear capacity (which helps in avoiding the undesirable and brittle shear failure), adequate amount and arrangement of transverse reinforcement also help to achieve large ductility/deformability by providing confinement to the core concrete and by restraining the buckling tendency of the longitudinal reinforcing bars.

6.2. Displacement based design

In inelastic seismic design, structural performance is measured by a combination of strength and deformation. The traditional way of design, which is also identified as force based, is to conceive a structure of the required strength, follow the prescribed detailing guidelines and show through analysis that it possesses the required level of ductility. In other words, strength is the primary design target and the displacement has always remained secondary. In the past two decades, increased emphasis has been placed on the use of structural deformations in structural design as a measure of seismic performance as opposed to the seismic forces within a structure. Such a shift in logic has been motivated by the fact that there exists a high correlation between deformations and material strains in structural components (which further define damage in structural components) as opposed to the relatively poor correlation exhibited between forces and material strains. This has led to a design method in which the required displacement is taken as the primary target and structures conceived to have the deformation capacity are checked if they meet the strength requirement. The shift towards such a displacement focused design procedure has been further emphasized by some of the current deficiencies in force based seismic design relating to the idealization of elastic stiffness characteristics of a structure and its elements. In particular, significant variability exists in the initial stiffness of reinforced concrete and masonry structures and also that such elastic characteristics are only relevant at low levels of seismic response until yielding, after which period elongation occurs. In addition, modal analysis which is used as a force based seismic design tool is elastic in nature and combines maximum response envelopes of different modes which may not have occurred at the same time; hence there are issues with the combination of modal contributions. Furthermore, stiffness (which is used to determine forces) is argued to be proportional to member strength, yet member strength is not known when the modal analysis is used for design, and an iterative process is not employed to reduce this inconsistency.

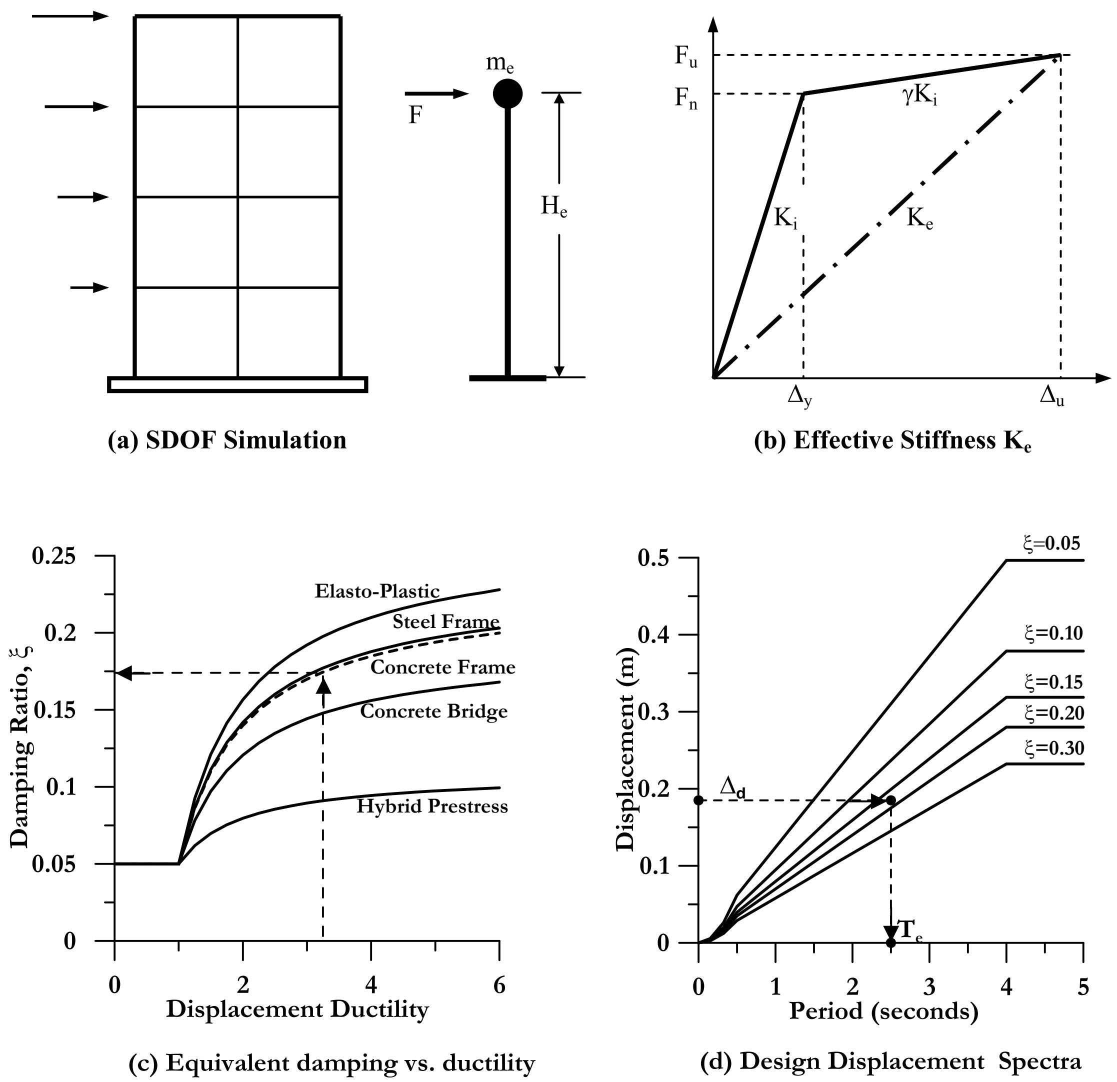


Figure 10: Fundamentals of direct displacement based design [Source: Priestley M.J.N., Calvi G.M., Kowalsky M.J. (2007). *Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia]

These deficiencies have led to the on-going development of direct displacement based design (DDBD). As shown in Figure 10, the design philosophy is based around the concept of idealization of a structure as a single degree-of-freedom system. Unlike the force-based methods which use initial stiffness, DDBD characterizes the structure by the secant stiffness at the maximum displacement. First, the maximum displacement for the design level of shaking is calculated using the material strain limits. Then, the level of displacement ductility is used to determine a level of equivalent viscous damping dependent on the structural form. A target level of design displacement is used in conjunction with a spectral displacement plot (for the level of viscous damping determined in the previous step) to determine the effective (i.e. secant) period of the structure. The period and the equivalent mass of the single-degree-of-freedom (SDOF) system are used to calculate the secant stiffness, which combined with the target displacement gives the lateral force for the SDOF structure. This force can then be distributed over the height of the structure to determine the forces for design of the constitutive elements which comprise the structure.

Although many regard DDBD as a significant improvement over conventional force-based design which is prescribed in current design codes, some concerns still remain on the DDBD methodology. Particular comments revolve around the use of the maximum displacement to determine the level of equivalent viscous damping and the secant stiffness of the SDOF system, while the second revolves around the appropriateness of the SDOF assumption for true multi-degree-of-freedom (MDOF) structures. The former comment is in regard to the fact that the peak displacement occurs for an instant during the dynamic response of the structure while it is used for the entire duration for the SDOF system used in determining the displacement spectra in DDBD. Hence, a more appropriate displacement to use for determining the period and damping would lie somewhere between the secant period based on the maximum displacement and the initial elastic period. The latter criticism of DDBD pertaining to the use of the SDOF representation of the MDOF structure, results from the fact that while a SDOF system may be appropriate for determining the displacement of the centre of mass and for assuming a global displaced shape, the SDOF approximation for the internal forces within the structure can be significantly incorrect, even for structures of moderate height where higher mode effects are generally not considered to be important.

7. Randomness and Uncertainty in Capacity and Demand

Estimated values of demand and capacity are likely to vary mainly due to randomness in the values of the variables used in their estimation and uncertainties in the method/model followed for their estimation. Randomness exists in both the demand on a structure and in its capacity to resist them. Earthquake motions are inherently random. Structural behavior is also affected by random variations in material properties and construction quality. Capacity is also affected by loading history and duration which are both influenced by the randomness of the excitation. Even with increased knowledge, there will be large randomness in both the excitation and response.

The uncertainty in the expected structural demand originates mainly from: (i) quantification of seismic hazard; i.e. seismology (such as uncertainty in the earthquake intensity during a given interval of time, and the various seismic hazard models and attenuation relationships available to interrelate the intensity with the return period and the source-to-site distance); (ii) variation in different characteristics (such as duration, frequency, response spectrum etc) of ground motions of the same intensity; (iii) structural characteristics (mass, damping, stiffness which affect the structural demand are difficult to be determined exactly); and (iv) structural analysis method (different analysis methods can be used resulting in different solutions of the same problem).

In modern seismic design, capacity can mean a combination of strength, deformability and energy dissipation ability. Although strength of elements (especially those controlled by flexure) can be predicted reasonably well, complexities and uncertainties arise due to slab contribution to the beam strength, panel zone deformation and bar pullout at the connection, contribution of shear, nonstructural components altering the behavior of structural elements etc. Design equations are usually conservative and provide lower bound values of the capacity. The lack of consistency in using such equations makes it hard to determine the failure mode of a system. Even when more rational relations are used to determine capacity, system behavior is still uncertain because most capacity estimates focus on elements rather than systems. It is often unknown whether the failure of one or two elements will lead to the failure of the system. Another source of uncertainty in capacity originates from the way capacity testing is conducted. Tests are often terminated at some arbitrary level of drift or ductility and not loaded until failure. Moreover, capacities corresponding to many intermediate limit states such as spalling, buckling etc are not well-documented, which makes assessment of these limit states difficult and somewhat subjective.

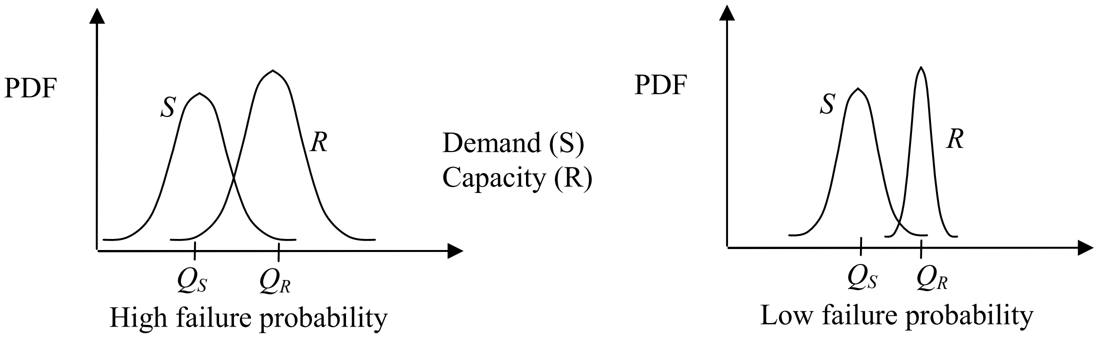


Figure 11: Effect of variations in capacity and demand in structural safety

Owing to the randomness and uncertainties outlined above, the actual capacity and demand may be different from those used in designing a structure. In the extreme ranges of demand and capacity, it is always possible that the seismic load exceeds ability of a structure to resist it without being damaged, partially or completely. In other words, despite following the design process as specified in the codes, it may not be possible to avoid failure completely. This argument is explained by comparing probabilistic distribution of capacity and demand as shown in Figure 11. In the Figure, the variation in demand and capacity and their expected values are shown. The area enclosed by the two curves in the middle amounts to the total probability of failure. In the left Figure the difference between the expected capacity and expected demand is greater than that in the right Figure (i.e. the conventional safety margin or factor of safety is bigger in the left Figure), but yet the probability of failure is greater in the left Figure. This summarizes why deterministic design blatantly fails to capture the risk. Merely by using a bigger factor of safety, the risk (or the probability of failure) cannot be reduced.

8. History of Seismic Design

8.1. Ancient structures

One question that often strikes in the mind of structural engineers when they see ancient structures (such as forts, temples, churches, towers, mosques, palaces etc) standing tall and firm through centuries is: how did these structures withstand so many earthquakes? Most of such structures are made of unreinforced stone/brick masonry. One common feature of such structures is that their load bearing components (columns, walls, beams etc) are very bulky in size. The owners of such monuments (usually the wealthy emperors) wanted to build long-lasting structures, but the builders did not have any clue how robust they needed to be to withstand the demand of nature. Therefore, they built huge and heavy structures to be sure of its longevity. Seismic resistance of such structures comes from the following sources:

1. Although unintentional, these structures are easily bigger than the sizes required for them to respond elastically during the design level earthquakes (in many cases, even the MCE).
2. These structures are so heavy and big that a huge lateral thrust is required to topple them. The resistance against toppling is the product of self weight and the eccentricity (i.e. distance between the centre of mass and point about which the structure topples). As both the weight and size are large, the resistance against toppling is insurmountable.
3. Due to the heavy weight, such structures are rigid and the acceleration response is similar to the PGA. Consequently, the resulting response acceleration is not magnified, which induces smaller (in comparison with the size) lateral seismic force.
4. The large compression force due to heavy self-weight ensures that the friction between different pieces stacked together is strong enough to take care of the shear demand.

Due to these inherent advantages, such structures have been able to withstand so many earthquakes in the past. Thus, the lack of knowledge of demand while conceiving these structures turned out to be a blessing in disguise.

8.2. Past: Era of evolution and advancements

Since the ancient days, earthquake engineering has been ever evolving. As explained earlier, the necessity of earthquake-resistant design was realized in the beginning of the 20th century. The very important concept of equivalent lateral force design, which is still intact despite all the advancements made in the field, was established when the seismic design guidelines were written for the very first time. Nevertheless, the value of the equivalent lateral design force was grossly underestimated during the initial period, which went through gradual increases throughout the last century. The design philosophy itself saw progression from the strength based elastic design method to the modern multi-objective performance based design. To keep up with the change in the design philosophies, the analytical methods used to estimate the strength (or lately to assess the required performances) also had to evolve. As a result, the simple calculations to estimate the elastic stresses for the first generation seismic design phase based on allowable stress method gradually matured into the modern time history nonlinear dynamic analysis which makes full use of the recent advancements made in the computational ability. Simpler and more elegant methods of analysis/design have also emerged through the years. Although the underlying design principle of "capacity greater than demand" has remained pivotal in all phases of seismic design throughout its evolution and advancement, the definition of capacity and demand and methods of estimating them kept on changing. Based on the historical progression of different aspects of seismic design, the journey so far can be classified into an era of evolution (first half of the 20th century) and an era of advancements (second half of the 20th century).

8.3. Present: Era of safe design

The modern method of inelastic seismic design ensures that a designed structure does not collapse even in a rare and large earthquake; i.e. it can be stated with high confidence that life safety will be achieved. Real case studies have also vindicated this aspect of modern seismic design; the fatalities in recent earthquakes have originated mainly from either non-engineered buildings made of adobe and unreinforced masonry or buildings designed and built before the modern versions of inelastic seismic design were in place. Many laboratory experiments have also proved that fatal collapse can be prevented if the capacity design approach is followed with careful detailing of the plastic hinges. Hence, it should not be an overstatement to name the modern era of inelastic ductile design as an era of safe design.

Nevertheless, capacity design (or any other inelastic seismic design philosophy) is based on allowing damage to occur in well detailed regions of a structure, although such damage may not be acceptable for some structures which have to be designed as high performance structural system. As observed during recent large earthquakes, the monetary consequences can be very high despite the fatality in the buildings designed with modern capacity design principles being very low (almost nil). The total financial loss comprises the cost of repairing the damage and replacing the damaged contents, the loss of profit due to the closure of the building during repair, and direct and indirect costs of human injuries. Ironically, despite huge financial implications were incurred during recent earthquakes, the design engineers had reasons to boast of being successful in taming the buildings to perform the way they wanted. Obviously, the society's wish and the designers' expectations from the buildings were not in agreement. While the designers have been focusing mainly on structural safety which they have succeeded to achieve, the society has found these buildings to be performing poorly in terms of economic efficiency, especially during moderate to large earthquakes.

8.4. Limitations of the present state of seismic design

As explained earlier, the current approach of using a factor of safety (or more than one factor) to account for the variability in capacity and demand does not result in absolute safety; and more importantly a higher factor of safety does not necessarily correspond to a lower probability of failure. Making decisions regarding earthquake risk is plagued by many uncertainties, which result from both the random (aleatory) nature of many earthquake phenomena such as the rupture dislocation process, ground motion propagation through the earth, response of the structure, damage due to structural response and loss due to the damage incurred. Uncertainties of a different nature (i.e. epistemic uncertainties) arise from our lack of knowledge or understanding of the complex process involved in the design, such as the structural properties used in modeling the structural system in a time history analysis, uncertainty in the frequency of occurrence of various levels of ground motion intensity etc. In order to make a systematic assessment of the seismic risk to a specific facility and then to make rational decisions regarding the mitigation or acceptance of such risk, this risk needs to be quantified first. Any risk assessment process adopted for this purpose requires consideration of the aforementioned uncertainties. Investigation of current design codes indicates that while there are some aspects of uncertainty treatment, many of them are not explicit, and therefore do not allow adequate quantification of seismic performance. Although ground motion intensity is treated semi-probabilistically via the use of uniform hazard spectra in current codes, seismic demands and structural response are treated entirely deterministically. In short, given that there are so many uncertainties involved in quantifying the capacity and demand, probabilistic statements such as “the probability of the calculated capacity exceeding the prescribed demand should be more than 0.9” would be more appropriate than deterministic statements such as “the calculated capacity should be greater than the prescribed demand”.

Previous single-criterion design approaches required the verification of building performance at an ultimate level of seismic hazard, which does not represent the effect of an earthquake that might occur in one of the nearby sources. Ground motion of any intensity within a feasible range could strike at any time, and a building should be able to satisfactorily respond to ground motions of all intensities. The modern multi-objective performance based design requires a building’s performances to be checked against three different hazard levels which are well spread in terms of severity to cover the feasible range, and hence the building’s performance at any hazard level can be interpolated using its performances at the three design hazard levels. Nevertheless, structural performances for a continuum of seismic hazard need to be explicitly taken into account to quantify seismic risk to facilitate informed decision making. Also as seismic risks are not assessed, the modern design does not provide a quantitative basis for owners/decision makers the compare design/retrofit alternatives.

In the current approach, performance is assessed at a component level and not system level; and the structural performance is deemed to be not satisfied if a single component fails. This does not allow for the fact that local failure of a building element does not necessarily result in global collapse due to the large redundancy in typical buildings. Thus, building collapse cannot be rationally defined by prescribed levels of displacement, but instead should be based on dynamic analyses where both sidesway collapse (due to P-delta effects) and shear and axial collapse (resulting in loss of vertical load carrying capacity usually leading to cascading failures) are taken into consideration. Almost all current codes consider the performance of buildings only with the limits on displacements in large ground motions mainly to prevent failure of building elements. Such displacement limits are typically well beyond those which cause significant damage to non-structural components in a building. Also, currently no codes account for maximum permissible floor accelerations which are more relevant (than the maximum displacement) to the safety of some non-structural components and contents in a building.

9. The Future: Era of Economic Design

As mentioned earlier, the modern inelastic seismic design, despite being able to avoid collapse and ensure life-safety, is not economically efficient. There is now a greater awareness and demand from clients and the public for engineers to provide economically efficient structural systems which incur minimum economic loss while preserving the life safety feature during large earthquakes. Seismic risk assessment of modern structures has suggested that a majority of the seismic risk/loss originates from the hazard posed by moderate earthquakes which occur more frequently than the big ones. This is because in modern design (such as capacity design) structures are expected to respond inelastically in moderate earthquakes, and in ductile structures inelastic response is always accompanied by damage. Although the cost to repair these minor-moderate damages in a single building may not be alarmingly high, the society as a whole has to bear a major financial burden in reinstating hundreds and thousands of such buildings each time a moderate earthquake occurs.

Up until now, one of the main objectives of earthquake engineering was to design, construct and maintain structures which, during earthquake exposure, perform up to the expectations and in compliance with building codes. In the future, this is likely to change to design and construct economically efficient structures which incur acceptable loss (up to the expectations of major stakeholders) in earthquake exposure. It is very likely that the next generation of seismic design methods will pay more attention to minimizing the financial consequences in addition to ensuring life safety. Such economically efficient seismic design methods are hereafter referred as “loss optimization seismic design (LOSD)”.

9.1. Design criteria for loss optimization seismic design (LOSD)

It is very likely that future design codes will become performance-based with less prescriptive restraints. The on-going trend is to use a matrix of discrete performance requirements for various levels of seismic intensity, as in the prescriptive multi-objective seismic design. However, unlike in the existing design which considers the performance of structural components only, performances of non-structural elements and contents will also be considered in the future because these contribute significantly (dominantly, in some cases) to the total financial loss incurred in a building during an earthquake. At the time of writing this chapter, research efforts are undergoing to develop performance-based frameworks which allow building performance to be measured in terms of repair cost, casualties, and number of days of downtime. These performance measures are more easily understood by non-technical stakeholders of buildings such as owners and insurers, and can be used in direct comparison with other hazards that affect the building. Evaluating and interpreting the risks in terms of such generic parameters lead to an easy process of efficient decision making.

Similar to current seismic design methods, a building designed based on LOSD approach also has to satisfy various performance criteria, but in addition to life-safety the criteria will also include minimization/optimization of financial loss in different levels of ground motion. The criteria will incorporate all forms of loss, commonly identified as three D’s: damage (“dollars” is also used by many), downtime and death. “Damage” covers damage to structural and non-structural components as well as to the content of a building, and “death” includes injury, too. The performance criteria for LOSD can be expressed in an RDI format, where R is the cost to repair the damaged components and to replace the damaged content (expressed as a percentage of the building value including contents), D is the number of closure days (i.e. downtime), and I is the injury vector specifying the probability of minor injury, major injury and death. An example of the required performances for different building categories in different levels of ground motions is shown in Table 2. In the Table, 0.1 is used to represent the minimum allowable limit because in probabilistic framework, it is not pragmatic to aim for an expected value of zero or for a value which has zero percentage probability of occurrence.

Table 2: Example of performance requirements in RDI format for buildings in different levels of ground motion in LOSD

Performance measures
Repair (R): Expected cost for component/content repair/replacement, μ_R (percentage of building value including contents) Downtime (D): Expected closure of the building for repair/replacement, μ_D (days) Injury (I): Expected likelihood of injury/casualty, μ_I (probability of minor/major injury and death)

		Allowable loss (Capacity)		
Ground motion intensity corresponding to	Expected loss (Demand)	Residential buildings	Commercial and office buildings	Emergency facilities
Frequently occurring earthquake (FOE), 50% in 50 yrs	Repair: μ_R	0.1%	0.01%	0.001%
	Downtime: μ_D	0.1 day	0.01 day	0.001 day
	Injury: μ_I (%)	0.1,0.01,0.001	0.01,0.001,0.001	0.001,0.001,0.001
Design basis earthquakes (DBE), 10% in 50 yrs	Repair: μ_R	10%	1%	0.01%
	Downtime: μ_D	10 days	1 day	0.01 day
	Injury: μ_I (%)	10,1,0.01	1,0.1,0.001	0.1,0.01,0.001
Maximum considered earthquake (MCE), 2% in 50 yrs	Repair: μ_R	No limit	10%	0.1%
	Downtime: μ_D	No limit	10 days	0.1 day
	Injury: μ_I (%)	50,10,0.1	10,1,0.01	1,0.1,0.001

Other performance criteria commonly used in current seismic design methods (e.g. immediate occupancy, reusability, collapse prevention, life safety) need not be explicitly specified in LOSD because the multi-level acceptable loss criteria implicitly ensure that these are catered for. For example, for the expected repair cost and downtime to be within their very small limits, a building should not undergo notable damage in a frequently occurring earthquake, which automatically ensures the immediate occupancy criteria. Similarly, once the criteria related to repair, downtime and injuries are imposed; the life safety and collapse prevention criteria are automatically taken care of. Threat to life safety comes mainly from severe damage or collapse, both of which will require the structure to be replaced. This will not satisfy the repair and downtime criteria in a maximum considered earthquake (MCE) except for residential buildings where severe damage requiring the building to be demolished is permitted as long as the probability of casualty is minimal. Minimization of damage will be achieved by LOSD, but response parameters such as drift and/or acceleration will not be explicitly restricted, unlike in current prescriptive design. The probabilistic LOSD approach allows structures to respond beyond the current limits as long as the three loss criteria are met. Innovative structural systems such as Damage Avoidance Design, which is based on self-centring rocking connections leading to possibly a larger response but less damage (thereby incurring less loss), will have little trouble in meeting the LOSD criteria, whereas they would not necessarily meet the maximum response based criteria of current seismic design.

9.2. Estimation of capacity and demand for LOSD

Note that in LOSD too, the design motto of "capacity exceeding demand" is intact; however, here the capacity is interpreted as the allowable/tolerable loss from repair, downtime and injury and the demand is the expected value of the corresponding form of loss a building may incur in an earthquake. As is normal in performance based design, the capacity (i.e. allowable losses) is structure specific (with respect to its importance and use). In this case, it is also client specific; e.g. a risk adverse client may set lower allowable limits. In addition, the capacity in LOSD is specific to stakeholders as well; e.g. insurers and owners will benefit from a stricter allowance for damage repair, whereas occupants will be more interested in specifying lower allowable injury/casualty probability and commercial tenants will prefer a building designed to have a lower downtime. On the other hand, the estimation of the demand (μ_R, μ_D, μ_I) requires a comprehensive probabilistic risk assessment methodology that takes into account all forms of uncertainties and randomness. Efforts are currently underway in earthquake engineering community to develop such methodologies. It may take a long time before simplified methods to estimate these demands (μ_R, μ_D, μ_I) could be prescribed in building design codes.

Assessment of seismic risk can be performed using: empirical approaches, expert opinions, or analytical approaches. Empirical methods are based on the use of raw data from previous occurrences of earthquakes and are therefore conceptually simple. However, the occurrence of earthquakes is so infrequent that the quantity of data available for estimation of seismic risk is small. In addition, generally available data do not correctly represent a required scenario. Expert opinion can be used in the absence of empirical data for the required building types. However, care must be taken while setting criteria for qualification of experts and detecting bias in their opinions. Analytical methods provide a framework which does not suffer from the deficiencies in the previous two methods. It should, however, be noted that analytical methods are described by many relationships, some of which may be defined or based on the use of empirical data or expert opinions.

9.3. Probabilistic seismic risk assessment framework

An analytical method for seismic risk assessment has been developed based on outcomes of recent research conducted at the Pacific Earthquake Engineering Research (PEER) Centre. This Performance Based Earthquake Engineering (PBEE) framework has recently been the basis for probabilistic risk assessment of different structural systems. PBEE is a tool to enable a designed facility to meet the needs (related to design, construction, maintenance, management and monitoring) of its owners and users, and it quantifies seismic risk which can be used to make informed decisions. The concept of PBEE is not new; it is similar to the multi-objective performance based seismic design but the performance assessment is conducted probabilistically to get a better appreciation of the risk involved. In PBEE, the owner selects the performance goals and the designer tries to achieve these performance goals. The PEER PBEE approach revolves around the integration of seismic hazard, seismic response, component damage and loss, as shown in Figure 12.

Figure 12: Schematic illustrations of the key steps in the PEER PBEE methodology

This risk assessment framework can be mathematically expressed by a triple integral equation:

$$f_a(dv) = \int\int\int G(dv \mid dm) \mid dG(dm \mid edp) \mid\mid dG(edp \mid im) \mid\mid df_a(im) \mid \quad (4)$$

in which f_a(x) = the annual rate of exceedance of (x); *im* = intensity measure (quantifying severity of ground motions); *edp* = engineering demand parameter (quantifying critical structural response); *dm* = damage measure (quantifying extent of damage); *dv* = decision variable (a damage related milestone whose occurrence, or non-occurrence, provides a basis for an important decision); and $G(x|y)=P(x>X|y=Y)$ is the conditional complementary cumulative distribution function (CCDF). Absolute value signs are required for the terms in Equation (4) because some of the derivatives of the CCDFs may be negative. In PBEE, it is common to use capital letters (IM, EDP, DM, DV) to represent the variables/parameters, whereas small letters (*im, edp, dm, dv*) are used to indicate specific values of these parameters. For different sources of seismic risk, the definitions of these parameters are given in Table 3.

Table 3: Variables for assessment of loss from damage, downtime and death

<table>
<tr><th rowspan="2"></th><th colspan="3">Damage</th><th rowspan="2">Downtime</th><th colspan="2">Death</th></tr>
<tr><th>Structural</th><th>Non-structural</th><th>Content</th><th>Injury</th><th>Casualty</th></tr>
<tr><td>IM</td><td colspan="6">Ground motion parameter (PGA, S_a etc.)</td></tr>
<tr><td>EDP</td><td colspan="2">Maximum drift</td><td>maximum acceleration</td><td colspan="2">maximum drift
maximum acceleration</td><td>maximum drift</td></tr>
<tr><td>DM</td><td colspan="2">extent of damage (cracking, yielding, collapse etc)</td><td>extent of disturbance (displaced, thrown, broken etc)</td><td>extent of damage/repair</td><td colspan="2">extent of damage and probability of collapse</td></tr>
<tr><td>DV</td><td colspan="3">repair/replacement cost</td><td>length of closure for repair</td><td>probability and extent of injury</td><td>probability of casualty</td></tr>
</table>

As can be seen in Table 3, differences between the three sources of loss exist in the interpretation of decision variables. The decision variable in Equation (4); i.e. *dv*, can be replaced by a desired quantity, such as direct repair cost or number of closure days, depending on which source of loss is being considered. Alternately, all of them can be converted to a dollar value if loss of income due to downtime is quantified and equivalent monetary value of a life is assigned based on local socioeconomic scenario.

Equation (4) gives the mean annual rate of exceedance of a decision variable *dv*. In assembly based vulnerability where performance of a structural system is derived by combining the vulnerabilities of its components, expected value of a decision variable is more useful than its rate of exceedance. For example, expected repair cost of a building can be reasonably approximated by combining the expected repair costs of its components, whereas combining the mean annual rates of exceeding a certain level of demand/damage of different components does not give any meaningful information. In order to compute the expected value of a decision variable, values of the decision variable across the entire range of probability must be integrated. For example, calculation of the expected annual loss (EAL) is shown below in Equation (5):

$$EAL = \int l_r \left| dP(l_r) \right| \tag{5}$$

where l_r = loss ratio (decision variable in this case) defined as the cost to repair a structure divided by the total replacement cost; and $P(l_r)$ = probability of loss ratio exceeding a specified value l_r. Rate and probability are numerically similar for small values (v<0.01) but deviate for larger values, and rate can be converted to probability using a temporal relationship. Using Poisson model for simplicity, the relationships between rate and probability and between their derivatives are given by:

$$P = 1 - e^{-f_a} \tag{6}$$

$$dP(im) = e^{-f_a} df_a(im) \tag{7}$$

where P = probability of exceedance in a year; and f_a = annual rate of exceedance. If $df_a(im)$ is replaced with $e^{fa}dP(im)$ and the decision variable dv is replaced with loss ratio l_r in Equation (4), it results in the probability of exceeding a loss ratio, $P(l_r)$. Hence, a complete expression for EAL can be calculated by substituting Equation (7) and Equation (4) into Equation (5) as:

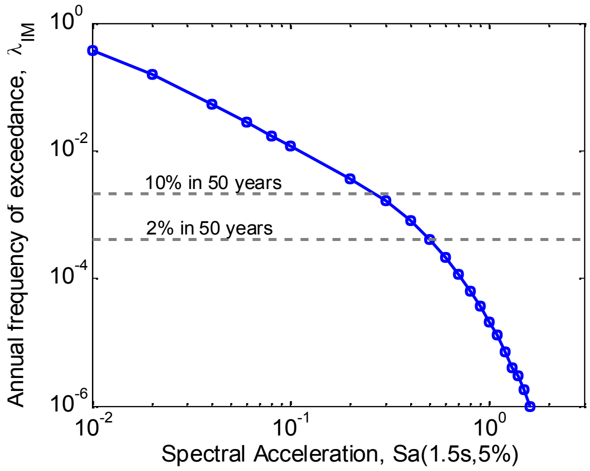
$$EAL = \int\int\int\int l_r dG(l_r \mid dm) \mid dG(dm \mid edp) \mid\mid dG(edp \mid im) \mid\mid e^{-f_a} df_a(im) \mid \tag{8}$$

Equation (8) provides the basis for calculation of expected losses from all three sources. Although EAL has been used here as a measure of a direct loss (i.e. cost to repair/replace damaged components and contents), it can easily be used as a general term to mean expected downtime or expected probability of injury/casualty if an appropriate decision variable representing downtime or injury/casualty is used instead of l_r. As is obvious from the equation, it is necessary to form probabilistic relationships between the multiple facets of the assessment process (i.e. hazard, response, damage, loss) to conduct a comprehensive seismic risk assessment. In particular, four interrelationships are required: (i) $f_a(im)$: annual probability of occurrence of earthquakes of a given intensity (also known as ground motion hazard relationship); (ii) *im-edp*: probabilistic structural response curves (obtained through special structural analysis methods); (iii) *edp-dm*: probabilistic response versus damage relationship (commonly known as fragility curves); and (iv) dm-l_r: probabilistic damage versus loss relationships (also called loss functions). Typical curves describing these four interrelationships are shown in Figure 13, and each of these facets is discussed in greater detail in the following paragraphs.

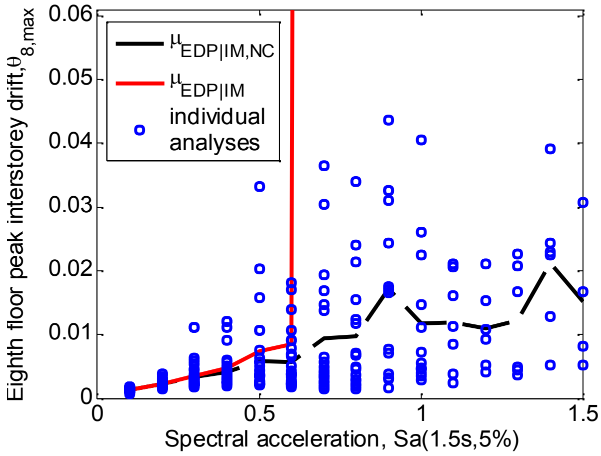
9.4. Probabilistic relationships required for loss estimation

Quantification of seismic risk of a structure in terms of probability or rate of occurrence (or exceedance) of a performance measure inevitably requires a relationship describing the rate of occurrence of various levels of ground motion intensity at the site where the structure is located. Such a relationship, typically determined by probabilistic seismic hazard analysis (PSHA), is commonly expressed in the form of a ground motion hazard curve (see Figure 13a). The ground motion hazard curve is determined from consideration of all earthquake sources located around the site, probability of various levels of rupture magnitude at each of these sources, and the effects of wave attenuation from the rupture location to the site. Continuing advancement in seismology is leading to more accurate estimates of the ground motion hazard.

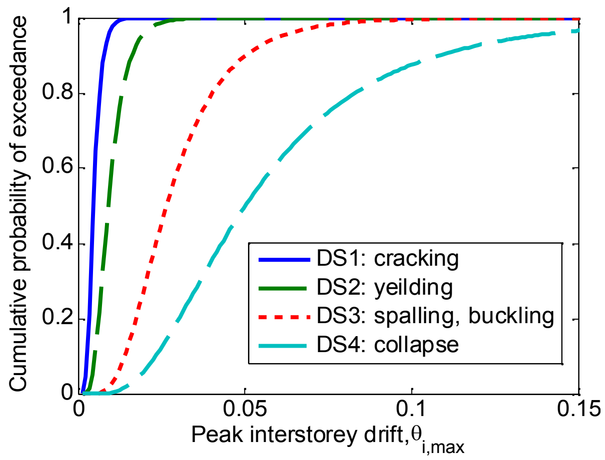
As previously mentioned, conventional seismic response analyses of structures are deterministic in the sense that no uncertainties are assigned to the analytical model of the structure, or to the adopted properties of the ground motion that the structure is subjected to. Ground motions are usually selected based on a prescribed level of intensity (e.g. the intensity of a ground motion with 10% probability of being exceeded in 50 years; i.e. 475 year return period), but no account is made for the fact that no two records with the same return period will induce the same response from a specific structure (commonly known as record-to-record randomness). In order to conduct a comprehensive assessment of the seismic performance of a structure, ideally an ensemble of ground motions should be used because it can then account for the aforementioned record-to-record randomness, while uncertainty in the structural model (e.g. uncertainty in the parameters of the constitutive models, damping, mass etc.) can be considered in various forms such as Monte Carlo simulation, first-order second moment analyses, or deterministic sensitivity methods. Currently the most rigorous method for accounting for these uncertainties in the structural response is through the use of Incremental Dynamic Analysis (IDA). IDA involves scaling of ground motion records to various levels of intensity in order to examine the full range of structural response, from initial elastic behavior through to global instability. An example of IDA results is shown in Figure 13b, where IDA curve for a ground motion record can be formed by connecting the response points corresponding to that record scaled at different intensity levels. The complete IDA results hence comprise as many IDA curves as the number of records in the adopted suite of ground motion. The multiple response values (i.e. *edp*) at each level of intensity (i.e. *im*), or vice versa, can be used to quantify the record-to-record randomness, while various methods are still being explored by researchers to account for modeling uncertainties.



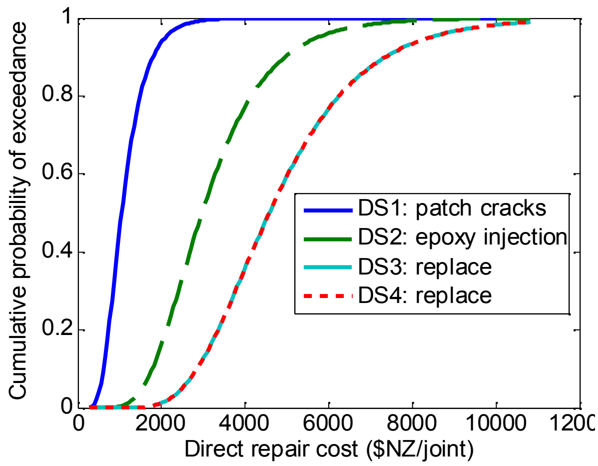
a) Ground motion hazard model: $f_a(im)$



b) Structural analysis: IDA curves: *im-edp*



c) Fragility curves: *edp-dm*



d) Loss functions: *dm-dv*(l_r)

Figure 13: Probabilistic relationships required for seismic loss/risk assessment

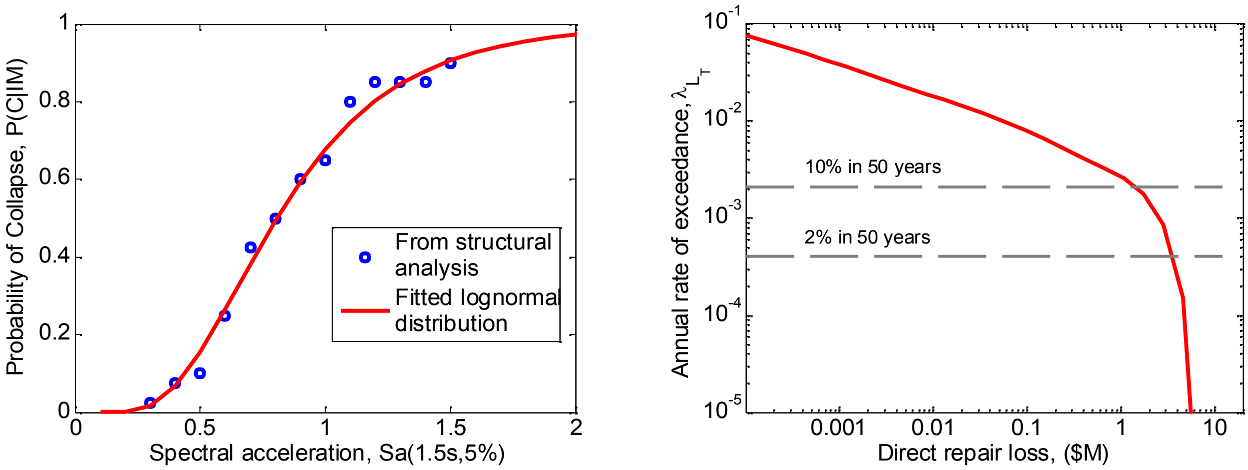
Although IDA provides a comprehensive method to assess uncertainties in seismic response analysis, some care must be taken with regards to the inputs (the ground motion suite and the structural model) and interpretation of the results. For example, scaling only modifies the amplitude of a ground motion while maintaining its frequency content or duration of the motion. Hence, scaling a single set of ground motions over a wide range of intensity induced obvious bias in the predicted response. Thus, care should be taken in selection of ground motion records for various levels of seismic hazard. In particular, deaggregation of the seismic hazard for a given rate of exceedance can be used to determine the magnitude of earthquakes originating at the source, the source-to-site distance, and the "epsilon" values (i.e. measure of deviation of a ground motion in comparison to the predicted mean at different periods) which provide the dominant contribution toward the site hazard. Similarly, care should also be taken to ensure that the constitutive models used in the analysis are representative of (to a desired level of accuracy) the material/element behavior over the full range of seismic intensity that the model will be subjected to during the IDA. In particular, the ability of constitutive models to capture strength, stiffness, and cyclic deterioration modes is crucial to adequately capture the phenomena of global dynamic instability.

Based on the dynamic response of the structure, various measures of seismic demand such as interstorey drifts and floor accelerations can be used to characterize the likely levels of damage incurred to all components (structural, non-structural, and contents) which comprise the structure. In this regard, components are typically assigned damage states which provide a discrete relationship between seismic demand and damage. Discrete damage states are used (as opposed to a damage continuum) because repair of damaged components will generally be a discrete process (e.g. a concrete structural element may require *patching* of minor cracks, *epoxy injection* for severe cracking, *replacement* for significant buckling or fracture of reinforcing bars). Various uncertainties in the level of demand which will cause a given level of damage are accounted for by using the concept of fragility curves, which represent the cumulative probability of exceeding a damage state as a function of seismic demand (see Figure 13c). These uncertainties (i.e. in *edp-dm* relationships) occur as a result of variations in material properties (e.g. variation in concrete and steel strengths) as well as the typical use of a single measure of seismic demand (i.e. peak inter-storey drift) whereas in reality the damage will be a function of the entire history of the loading (i.e. the number of smaller loading cycles and their associated magnitudes). Fragility curves for component and content damage are generally developed based on a combination of laboratory experiments and analytical methods.

Similar to fragility curves which provide relationships between seismic demand and component damage via the use of damage states, relationships between different damage states and loss are also required to complete the loss assessment process. These relationships are generally expressed in the form of loss functions, as shown in Figure 13d. The loss functions give the distribution of loss for the occurrence of a given damage state, and account for uncertainties in contractor's costs, times of completion and the effects of demand surge (i.e. temporary inflation of the cost due to high demand following an earthquake). Separate loss functions can be used for loss due to direct repair cost of fixing damage of different components and for the downtime required to repair the damage. Although a similar approach can be used for human injuries and casualties, for simplicity casualties are commonly considered to occur only in the case of global structural collapse. Although significant progress has been made so far in modeling and development of fragility curves and loss functions for structural, non-structural and content damage, modeling of fragility and loss functions for downtime and injury/casualty related losses is still in infancy. Nevertheless, in several buildings these indirect losses can have far larger consequences than the direct repair cost of the damage. Therefore, the development of fragility and loss functions for these indirect losses need more attention in future.

9.5. Quantification of risk measures and their application in decision making

By combining each of the aforementioned relationships it is possible to provide several different quantifiable measures of seismic performance or seismic risk. Conducting two or more integrations among the four integrals in Equation (8), seismic risks can be interpreted in different useful and informative ways. For example, by tracking the collapse points of different IDA curves, the probability of collapse at different ground motion intensity level can be represented in the form of collapse fragility curve, as shown in Figure 14a. Combining the ground motion hazard relationship with the probabilistic structural response relationship (i.e. the IDA curves), demand hazard curve can be generated, which provides the mean annual rate of exceedance of an *edp* or a likely value of EDP in a scenario event. Similarly, combining the demand hazard curve with the fragility and loss functions leads to the loss hazard curve (shown in Figure 14b) which plots the annual rate of exceedance of different values of loss and can also be used to obtain scenario loss in an earthquake of a given annual probability. The area under the loss hazard curve gives the EAL which takes into account the consequence of all possible earthquakes. Using the aforementioned four relationships in appropriate forms similar loss hazard curves can be generated for damage, downtime, injury and casualty, and the corresponding measures of seismic loss can be estimated by using this loss assessment framework. Such quantified measures of direct and indirect losses (i.e. the loss due to direct repair costs, injuries and casualties, and business disruption/downtime) can be readily used by stakeholders to make rational decisions.



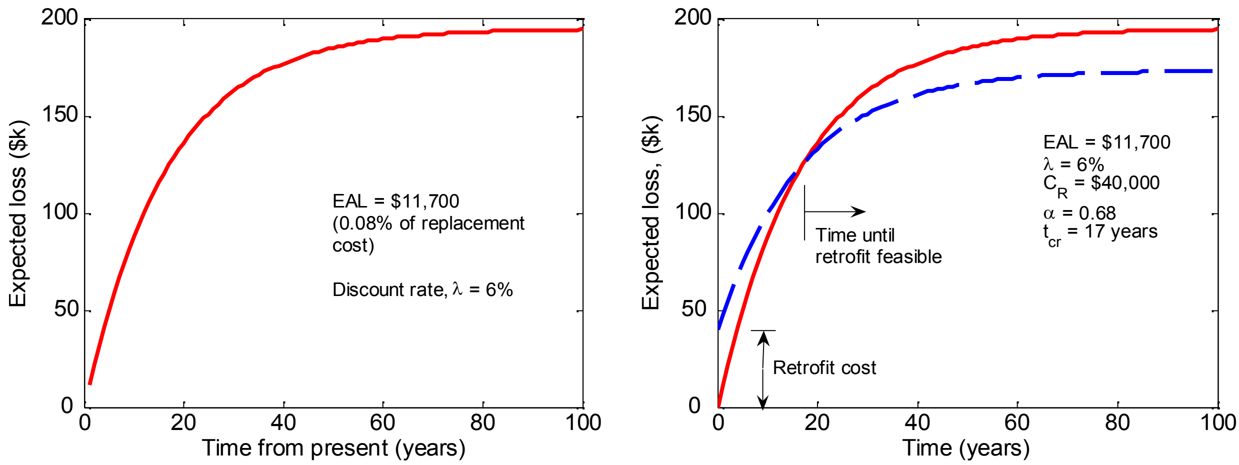
a) Collapse fragility curve b) Loss hazard curve (Area: EAL)

Figure 14: Assessment of collapse probability and EAL via probabilistic risk assessment

As is obvious from the preceding explanation, quantification of seismic risk based on this methodology requires significantly more rigor than is currently employed in seismic design and assessment of conventional structures. However, this has been made possible by the recent advances in seismology, structural engineering and social sciences. As mathematical operations in such a probabilistic loss assessment process are complex, automated computer based tools are also being developed for this purpose. In particular, PACT (Performance Assessment Calculation Tool) developed by the Applied Technology Council (ATC) and SLAT (Seismic Loss Assessment Tool) developed at University of Canterbury are currently available computer based tools known to the author. Using such a tool and a structural analysis program to conduct IDA, probabilistic seismic risk assessment of complex structural systems comprising several components can be performed if appropriate fragility and loss functions of the different components are available. Results can be extracted in several insightful forms from these computer based tools. For example, it can be known which storey in a multi-storey building contributes the most to the total risk/loss and which component (structural, non-structural, contents) in that storey will be damaged the most in a scenario earthquake. Similarly, the most vulnerable element (columns, beams, connections, slabs, walls etc) can also be easily identified. Also, the change in the overall risk by altering the design (and hence the performance) of different elements at different storeys can also be investigated. All these information is extremely useful in LOSD of new buildings and in formulating risk mitigation strategies for existing buildings.

The expected value of annual loss; i.e. EAL, which accounts for the direct loss due to damage of components and contents is a very handy parameter in loss based decision making. Indirect losses due to downtime and injury/death can be considered as separate entities, or alternately they can be combined with EAL by evaluating the loss of income (in dollars) due to the closure of a building, and by assigning appropriate dollar values for minor injury, major injury and casualty. As shown in Figure 15a, expected value of direct maintenance related loss over the life time of a building can be computed by accumulating the net present value of the EAL for each year using a projected discount rate. Adding the expected lifetime loss with the initial cost of a building gives its life-cycle cost. EAL and life-cycle cost together provide a very sound basis for deciding the viability of different design alternatives or retrofit solutions; an example is illustrated in Figure 15b.

In this case, the EAL of a building is calculated to be $11 700. If the building is retrofitted by installing viscous dampers (which costs $40 000) to primarily reduce acceleration demands in the structure due to ground motion shaking, the EAL can be reduced to $8 000. If a constant discount rate, λ,



a) Expected loss in net present value over time b) Decision on economic viability of retrofit

Figure 15: Application of EAL in life-cycle cost estimation and retrofit strategy

is assumed for the future, the net present value of the expected loss over time, E_L, can be computed by:

$$E_L = \frac{\left(1 - e^{-\lambda t}\right)}{\lambda} EAL + C_R \tag{9}$$

where t is the time in years; and C_R is the retrofit cost. Then by equating Equation (9) for the original building ($C_R = 0$) and the retrofitted building, the time after which the retrofit is economically feasible can be derived as:

$$t_{cr} = \frac{-1}{\lambda} \ln\left(1 - \frac{\lambda}{1-\alpha} \frac{C_R}{EAL}\right) \tag{10}$$

where α is a parameter indicating the reduction in EAL due to the retrofit; in this case $\alpha = 0.68$ (i.e. 8 000/11 700). Equation (10) yields t_{cr} = 17 years as the critical time for the economic viability of the proposed retrofit method (Figure 15b). In other words, the service life of the structure should be greater than 17 years in order for the retrofit to be financially beneficial. In this example, EAL (which represents direct maintenance costs) is used as the performance measure of interest. However, in general the performance measure to be used will depend on the perspective from which the decision is made. For example, the performance measures an owner and an occupant will choose to make decisions will be different, with the owner being principally interested in minimizing structural damage and business downtime, while the occupant is mainly interested in minimizing contents damage and avoiding injury/casualty.

10. Acknowledgements

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Author's Biography

Rajesh P Dhakal received a Bachelor degree in Civil Engineering from Tribhuvan University, Nepal in 1993 with a gold medal, a Master of Engineering degree with specialization in Structural Engineering from the Asian Institute of Technology (AIT) in 1997 with another gold medal, and a PhD in Civil Engineering from the University of Tokyo in 2000. Currently, he is an Associate Professor in the Department of Civil and Natural Resources Engineering at the University of Canterbury, where he has been working since 2003.

Dr Dhakal's teaching and research involvements are in the areas of Structural and Earthquake Engineering. He has authored more than 170 technical papers in the areas of reinforced concrete, earthquake engineering, and structural fire engineering. He is a member of several national and international professional organizations and has served as a reviewer for a number of International journals in Structural and Earthquake Engineering disciplines. Currently, he is an Associate Editor for the ASCE Journal of Structural Engineering, and an Editorial Board member of four other International journals in Civil Engineering.

Dr Dhakal is serving in the management committee of the New Zealand Society for Earthquake Engineering (NZSEE) and he is also a Chartered Professional Engineer (CPEng) in NZ. He was extensively involved in evaluating buildings in Christchurch after the 2010 September and 2011 February earthquakes, and has since been actively dissipating information on performance of buildings in these earthquakes to the professionals and public through different media. Along with other fellow Engineers, Dr Dhakal has been awarded the IPENZ President's award (Fulton Downer Gold Medal) for an outstanding engineering service to the community. He has also received four best paper awards, three best academic performance awards, and two best researcher awards including the prestigious Ivan Skinner award in 2007 for the advancement of earthquake engineering research in NZ.