Calibration of Seismic Design Codes using Loss Estimation

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SUMMARY
A radical departure from the current format of seismic design codes is presented in this paper. The basic principle of the proposal made herein for the calibration of codes for performance-based design is that the matching of seismic loading and performance levels should be based on a quantitative comparison of the incremental costs of adding seismic resistance and of the associated losses that can thus be avoided. The outcome of these analyses is an optimal map of design levels that can be used for regular buildings, which entirely circumvents the need to present seismic actions in the design code: the code need only present a zonation map, and specify the corresponding design level to be applied in each zone.

Keywords: seismic design, calibration, loss assessment, cost-benefit

1. INTRODUCTION

1.1. Performance-Based Seismic Design

Seismic design codes for the earthquake-resistant design of structures are fundamental for the mitigation of seismic risk. Codes provide guidance for engineers (who may not have specialist training in earthquake engineering) on analysing the effects of earthquake ground motions on structures, and on the required configuration and detailing for improved seismic performance. The crucial elements of a design code are the definition of the seismic actions to be considered in the design and the minimum levels of force and displacement that the structure should achieve under the imposed actions.

The basic performance objective in most seismic design codes has always been related to ensuring the life safety of the building occupants, generally through collapse prevention. The ground motions for which life safety was to be ensured by the design, at least until recently, have been based on the same criterion in almost every seismic design code throughout the world: the 5%-damped spectral acceleration with a return period of 475 years (Bommer and Pinho, 2006).

The experience of the 1994 Northridge earthquake and the 1995 Great Hanshin [Kobe] earthquake demonstrated that both economic losses and human casualties could be considerable, even if the no-collapse objective had been met for many structures (Bommer and Pinho, 2006). Performance-based seismic design (PBSD) formalises the approach of citing multiple objectives for structures to withstand minor or more frequent levels of shaking with only non-structural damage, whilst also ensuring life safety and no-collapse under severe shaking (ATC, 1978).

These objectives define the limit states, which describe the maximum extent of damage expected to the structure for a given level of ground motion. Many seismic codes define limit states or performance objectives according to the return period (TR) (or probability of exceedance) of a given level of ground motion. On others, serviceability and operational objectives may be assumed implicit, and are not necessarily afforded an explicit return period. Despite the diversity of definitions of different limit states, there is clear persistence in the adoption of 475 years as a basis for "life safety", although several codes have recently begun to adopt 2475 years as the return period for the no-collapse criterion, albeit subsequently rescaled to incorporate an assumed inherent margin of safety against collapse (see e.g. Weatherill *et al*., 2010).

1.1. Risk-Based Approach to Design

The 2009 revision to the NEHRP Provisions introduces a new conceptual approach to the definition of the input seismic action. The seismic input (maximum considered earthquake) is modified by a risk coefficient (for both short and long periods). This coefficient is derived from a probabilistic formulation of the likelihood of collapse (Luco *et al*., 2007). These modifications change the definition of seismic input to that which ensures a more uniform level of collapse prevention.

The risk-targeted approach presented by Luco *et al*. (2007) only considers the no-collapse limit state. It remains to be seen how this can be adapted to consider serviceability and damage limitation requirements. The degree of seismic detailing required to meet performance based objectives is affected by the behaviour of the structure at lower intensities. A longer term objective for performance based seismic design may be to consider the relative cost-benefit that a given level of detailing produces. Such an approach has been proposed by Bommer *et al*. (2005), who describe an iterative procedure to determine the cost versus benefit using displacement-based earthquake loss assessment. This approach would allow for designers to consider losses at other limit states besides collapse. This paper extends the paradigm shift proposed by Bommer *et al*. (2005) for the calibration of design codes, and applies it to a case study application in Turkey.

2. CODE CALIBRATION USING LOSS ASSESSMENT

2.1. Introduction

The basic principle of the proposal made herein for the calibration of codes for performance-based design is that the matching of seismic loading and performance levels should be based on a quantitative comparison of the incremental costs of adding seismic resistance and of the associated losses that can thus be avoided. Such as cost-benefit approach to code calibration requires that the structural parameters in the vulnerability model can easily be adapted to model increasing levels of seismic resistance, and thus enhanced design criteria. The decisions regarding investment in engineering design to achieve specified levels of seismic resistance, whilst informed by engineering seismologists and earthquake engineers, must ultimately reside with owners and regulators.

The procedure proposed herein has similar components to a methodology under development at the PEER Centre (e.g. Porter, 2003) for PBSD: the loss in terms of costs, casualties and downtime is considered for increasing levels of hazard from a cost-benefit viewpoint in order to define the loading-performance couple used in the design. However, an important difference between the method proposed herein and that by PEER is that the latter has not been derived for the calibration of design codes. The PEER approach is applicable for building-specific design and thus does not consider the convolution of hazard and vulnerability on an urban scale, which is the basis of the methodology proposed herein.

2.2. Iterative Loss Modelling for Different Seismic Design Levels

The first step in the proposed framework requires the building stock that will be located within a given area of interest to be modelled in terms of the number and location of different construction types and of different numbers of storeys, i.e. as building classes.
The next stage is to assign to each building class different levels of earthquake resistance through increments in stiffness/ductility over and above those resulting from non-seismic design according to the relevant building regulations. The DBELA procedure (e.g. Crowley *et al*., 2004; Bal *et al*., 2010)

is used herein; since it is based on the mechanical properties of structures, it is well suited to modelling an incremental improvement of the design levels. The basic level of non-seismic design defined by the relevant building codes is design level SC1, and each incremental improvement represents another SC_k.

Each level of building stock resistance is then subjected to the model of the seismic hazard in order to estimate the mean damage ratio (MDR) at different annual frequencies of exceedance (AFOE) for each location in the area under consideration. Loss (or MDR) exceedance curves for various levels of seismic resistance for a given construction type, as presented in Figure 2.1, can in this way be constructed. It should be appreciated that Figure 2.1 is illustrative and in practice a much larger number of SC_k levels is envisaged for actual code calibration studies.

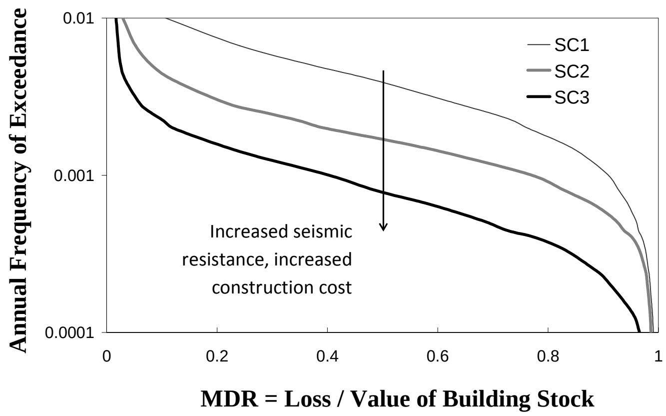

Figure 2.1 Illustrative plots of mean damage ratio versus annual frequency of exceedance for different levels of seismic resistance (SC_k) for one construction type.

2.3. Cost-benefit Basis for the Optimum Seismic Resistance

The next part of the process requires the decision makers to choose the level of seismic protection that a society is willing to pay for. There are a number of ways in which the cost-benefit of each SC_k can be presented. The simplest way of determining the optimum seismic resistance (in terms of the balance between investment and losses avoided) could be to look at a single scenario, such as the repetition of an important historical earthquake, and compare the losses incurred to each SC_k under this event with the corresponding cost of providing that level of seismic capacity.

A more robust procedure would be to consider all possible sources of ground motion at the site and to create a MDR exceedance curve, as has been presented in the previous section. Figure 2.2 shows the direct loss exceedance distribution curves which might be obtained from the MDR exceedance curves by assigning a nominal cost of 10,000 units to providing the SC2 level of resistance and 12,000 units to providing that of SC3. Additional losses such as those due to downtime for industrial and commercial activities and the impact this has on the regional economy could also be included in these curves.

The annual average loss (AAL) is often used by the insurance and reinsurance industries to enable them to set annual premiums. The AAL is the expected value of a loss probability curve and can be thought of as the product of the loss for a given event l ($Loss_l$) with the annual probability of occurrence of event l (OP_l), summed over all events:

$$AAL = \sum_l (OP_l \cdot Loss_i) \tag{2.1}$$

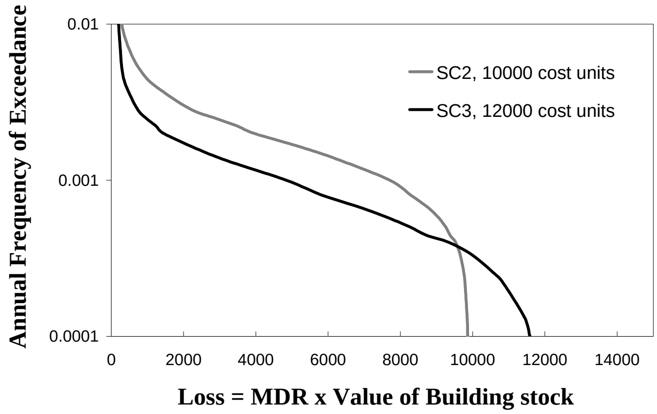

Figure 2.2 Loss exceedance probability curves for the acceptable levels of seismic resistance (taken from Figure 2.1) using a nominally assigned cost for each SC_k, as indicated in the plot

This can be directly compared with the cost of implementing each SC_k in order to attain the optimum seismic capacity level. In Figure 2.2, the loss of SC3 is higher than SC2 at low probabilities of exceedance (corresponding to high levels of MDR) because the higher the value of the building stock, the higher the losses for a given MDR. However, for the same example the AAL of SC2 is actually higher than SC3.

Alternatively, the loss exceedance probability curves can be used to create total cost exceedance probability curves. These curves combine the cost of each SC_k with the direct cost of damage (i.e. loss). For a specified level of cost, the annual frequency of exceedance can be obtained from the plot for each SC_k and a decision can be made as to whether this probability is acceptable. The higher cost of the superior seismic resistance leads to higher total costs at low exceedance frequencies, when the majority of the building stock is predicted to be heavily or completely damaged. In addition, at high frequencies of exceedance when the MDR is low, the total cost of the more enhanced seismic capacity is greater. However, at intermediate annual probabilities of exceedance, the seismic resistance with the lowest total cost (i.e. the optimum SC_k) will depend on both the shape of the loss curve and the cost of the seismic resistance.

It is proposed herein to use a cost-benefit factor that is the sum of the initial construction investment (cost) and the average annual loss (or the average loss within a given time span) (benefit) as a basis for making a decision on whether the additional investment to achieve SC3 as opposed to SC2 is justified.

$$\text{C/B Factor} = \text{Cost}(SC_k) + \text{AAL}(SC_k) \qquad (2.2)$$

As the levels of hazard will vary throughout the area of interest, not all locations will need to have the same level of SC_k, hence the SC_k with the lowest aforementioned cost/benefit factor should be assigned to each grid cell in the model.

2.4. Defining the Minimum Level of Seismic Resistance for Life Safety

In order to ascertain the minimum degree of seismic resistance required for each construction type, an uncomfortable but necessary decision must be taken by politicians, planners and code drafters regarding the tolerable levels of casualties, injuries and persons rendered homeless as a result of an earthquake ground motion with a specified annual frequency of exceedance. This will not necessarily be stated as a single number of casualties for a single return period, but could be expressed as combinations such that a death toll of 1,000 could perhaps only be “tolerable” for a return period of, say, 100 years, whilst for a 10-year return period, the tolerable death toll could be limited to perhaps 50.

The number of fatalities can be taken as a proportion of the expected occupancy of the given construction type, and can be assumed to be controlled by the complete damage limit state (LS3). Therefore, the tolerable threshold can be reduced to a specific proportion of the exposed building stock reaching and exceeding LS3. Hence, once the most cost-effective SC_k has been assigned to each location within the area under consideration, it should be checked to see if higher exceedance frequencies/probabilities than allowed at the threshold damage level are estimated; if so, the SC_k should be replaced with the next most cost-effective SC_k that meets the threshold.

The minimum level of required resistance and consequent investment in the building stock can thus be defined. The same procedure could also be applied to impose any number of risk criteria that will exclude any SC_k that violates the condition: limiting the proportion of buildings failing the extensive damage limit state (LS2) for specified return periods could effectively control both injuries and homelessness, and if applied to industrial and commercial buildings could also, in a crude manner, control downtime due to disruption.

2.4. Optimum Design Levels

The outcome of these analyses will be an optimal map of SC_k levels. This now entirely circumvents the need to present seismic actions in the design code: the code need only present a zonation map, and specify the corresponding design level to be applied in each zone. These design levels will be specified essentially in terms of the required levels of stiffness, strength and ductility capacity in buildings. There is thus no requirement to present a simplified response spectrum to represent the earthquake actions in the code and much less is there a requirement to anchor this spectrum to an arbitrarily selected return period.

3. CASE STUDY APPLICATION OF THE CALIBRATION METHOD

3.1 Case Study Area

The calibration method presented in Chapter 2 has been applied to the northwest area of Turkey. This choice was due to two main reasons: the availability of a reliable PSHA input model and the fact that this region comprises zones which range from low to high seismicity. A building class of mid-rise reinforced concrete buildings has been considered for this illustrative example. All analyses presented herein have been done with the OpenQuake software (http://openquake.org), that is currently under open source development as part of the Global Earthquake Model initiative (www.globalquakemodel.org). Further details are provided in the OpenQuake Book (GEM Foundation, 2011).

The seismic hazard input data utilized herein comes from a preliminary seismic hazard model developed for Turkey (Demircioglu *et al*., 2008). The PSHA model consists of a seismic source model based on two source typologies: area and faults. Faults are utilized to model large magnitude events (i.e. with moment magnitude $Mw \geq 6.7$), while area sources describe distributed seismicity for $Mw \geq 5.0$. Area sources are employed for two different purposes: to model large-scale background seismicity ($5.0 \leq Mw \leq 6.5$), as well as seismicity around faults (that is, events not occurring on the fault plane but within its neighbourhood). Earthquake ruptures inside area sources are modelled as points, while on fault sources ruptures are modelled as rectangles, whose dimension (length and width) are derived from the Wells and Coppersmith (1994) magnitude-area scaling relationship.

Faults are assumed to be vertical (dip angle equal to 90 degrees) with a strike-slip mechanism (rake angle equal to 0 degrees according to the Aki and Richards convention). Fault surfaces extend from 0 to 15 km depth. Area sources are associated to an average hypocentral depth of 3 km. Both faults and area sources occurrence rates follow a truncated Gutenberg-Richter magnitude frequency distribution. The OpenQuake engine has been used to calculate stochastic event sets and associated ground-motion fields, which were used in the calculation of the loss curves (as per the methodology described in the

OpenQuake Book).

3.2. Vulnerability Functions for Varying Design Levels (SCk)

Three levels of design have been considered in this case study:

- Basic seismic design (SC1);
- First level of improved seismic design (SC2);
- Second level of improved seismic design (SC3).

The first task of this study was to derive fragility curves for three different building designs with increasing seismic capacity. These fragility functions were derived using the DBELA fragility function approach (see Silva *et al*., 2012) using the parameters and assumptions presented in Table 3.1 and Table 3.2. As can be seen from Table 3.2 the improved design levels lead to an increase in both stiffness and ductility. Such changes caused a slight decrease in the displacement demand and a considerable increase in the displacement capacity, which consequently reduced the overall likelihood of suffering heavy damage or collapse.

Table 3.1 Material and geometric properties of the base building design (SC1)

Parameters	Mean	CoV	A*	B*	Distribution
f_y (MPa)	470	0.16	250	720	Truncated Normal
$\varepsilon_{concrete}$ (LS2)	0.0045	0.51	-	-	Lognormal
$\varepsilon_{concrete}$ (LS3)	0.015	0.51	-	-	Lognormal
ε_{steel} (LS2)	0.015	0.25	-	-	Normal
ε_{steel} (LS3)	0.06	0.25	-	-	Normal
Storey height	2.84	0.03	2.50	3.30	Truncated Lognormal
Beam length	3.37	0.38	1.0	7.5	Truncated Lognormal
Beam depth	0.48	0.14	0.3	0.6	Truncated Normal
Column depth	0.49	0.28	0.4	1.2	Truncated Lognormal

***A and B are the minimum and maximum boundaries, respectively, of the suggested distribution**

Table 3.2 Assumed modifications to stiffness and ductility

Building design	Period/height relationship	Increase in ductility
SC1	0.080H	0%
SC2	0.075H	40%
SC3	0.070H	80%

The fragility functions were combined with consequence functions in terms of economic losses and fatalities (see e.g. Bal *et al*., 2010) to produce vulnerability functions. Figure 3.1 presents the resulting vulnerability models for economic losses (as a ratio of the repair cost versus the replacement cost) and human losses (as the ratio of number of casualties to the total building occupants).

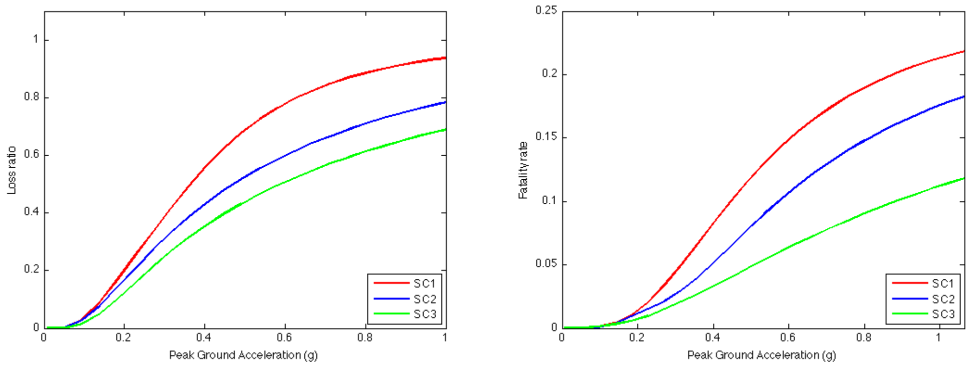

Figure 3.1 Economic vulnerability model (left) and fatality vulnerability model (right)

3.3. Loss Exceedance Curves for each SCk

In the first part of this exercise, only economic losses are considered. Using a probabilistic event-based risk calculator (as described in the OpenQuake Book), a hundred realizations of the seismicity (each with a 50 years time span) were used, leading to about 3300 ground motion fields. Due to the small spatial resolution (0.05x0.05 decimal degrees which for this latitude represents a distance of about 4 km), it was possible to take into account the spatial correlation of the ground motion, using the Jayaram and Baker (2009) model. Figure 3.2 shows a hazard map in terms of PGA for a probability of exceedance of 10% in 50 years, which has been obtained from the ground motion fields.

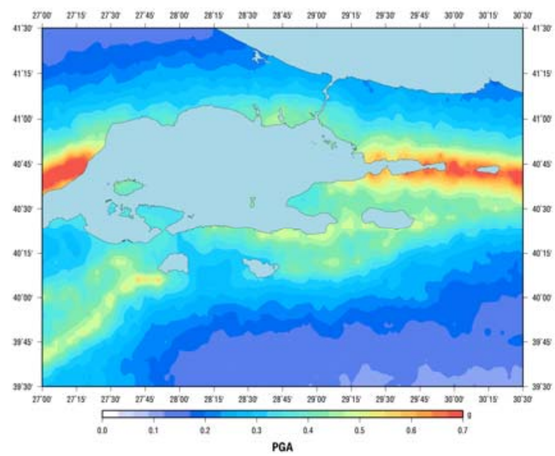

Figure 3.2 Hazard map for 10% probability of exceedance in 50 years

Once the seismic input was available, an aggregate loss exceedance curve was computed by placing a mid-rise reinforced concrete building in each location of the grid. Taking into account the aforementioned spatial resolution and excluding the areas where only water exists, a total of 2187 locations were considered. The same value of the building class was assumed in each grid cell, assuming that that probability of constructing in each grid cell is uniform; such assumption can be easily modified as a function of future land use planning. The base design level (SC1) was assumed to be 1 in each grid cell, whilst SC2 was assumed to cost 1.05 (i.e. 5% higher than SC1) and SC3 was assumed to cost 1.10 (i.e. 10% higher than SC1).

Three separate analyses were carried out, each time assuming a uniform distribution of the building design level throughout the region of interest. The aggregate loss exceedance curves across the whole region for the three design levels are presented in Figure 3.3.

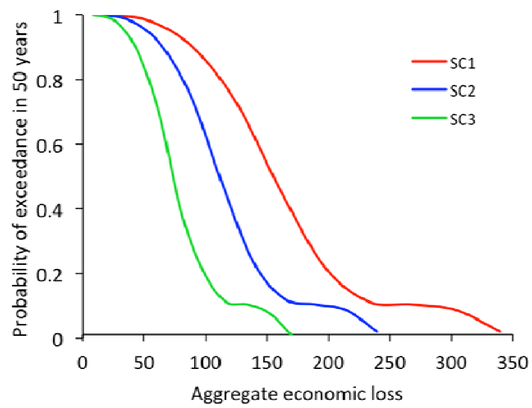

Figure 3.3 Aggregate (total) loss curves for differing seismic design levels based on the assumed unit costs

3.4. Cost-Benefit Analysis for Optimum Design

If only the aggregate loss exceedance curves were to be taken into account then the decision of which design level should be chosen would be straightforward: the most improved design (SC3) produces lower economic losses. However, as discussed in Section 2.3, the initial cost to build such a seismic resistant structure should also be taken into account. Thus, for each location, a cost-benefit factor comprised of the initial cost plus the mean expected loss in 50 years was computed. A time span of 50 years was used as this is the commonly assumed design life of residential buildings. The latter value is derived by integration of each loss curve at each grid cell location. Again, these calculations were performed assuming the three different design levels (SC1, SC2, SC3). As described in Section 2.3 the design level with the lowest cost-benefit factor was selected for each grid cell. Figure 3.4 shows the distribution of optimum design levels across the region of interest.

By comparing the distribution of building typologies with the hazard map presented previously in Figure 3.2, it can be concluded that in general for zones where medium to high ground motion values are expected, the usage of an improved seismic design is more economical, despite the fact that this typology has a higher initial cost. However, the regions of highest hazard lead to the lowest seismic design being the most cost effective as the losses are high regardless of the design level, and the combined cost-benefit factor is lowest for the most economical design. As expected, in regions where the seismicity is much lower, the minimum design level was the most cost effective. The base design (SC1) provided the most economic solution in 45% of the sites whilst SC2 and SC3 proved to be the most profitable choice in 27% and 28%, respectively. The aggregate cost-benefit factor has been computed for the 4 scenarios (SC1 in all cells, SC2 in all cells, SC3 in all cells and optimum SCk for each cell) and as expected, the latter scenario (with the optimized distribution of design levels) proved to be the most cost-effective solution, as presented in Figure 3.4.

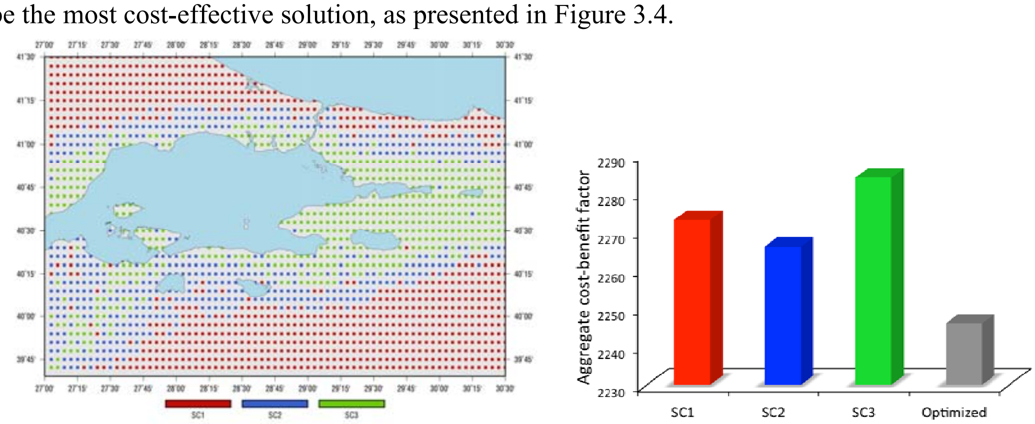

Figure 3.4 Optimized seismic design level distribution (left) and aggregate cost-benefit factor for each design level (right)

3.4. Minimum Level of Seismic Resistance for Life Safety

The results presented in Figure 3.4 do not take into account the performance of the buildings in terms of fatalities, which will have an influence on the minimum acceptable levels of design. A threshold on the number of casualties, for a certain return period, has been assigned for the purposes of this application. A loss curve was computed for the building class in each grid cell (using the vulnerability model presented in Figure 3.1), and the percentage of fatalities corresponding to a 2% probability of exceedance in 50 years (return period of 2475 years) was extracted and checked against an arbitrary and demonstrative "allowable" human loss of 5% of the occupants. A new distribution of design levels was created, assigning the building design that was closest to this threshold, but that did not exceed it, at each location. In the case where none of the design levels were complaint with this requirement, it was decided to attribute the safest typology, SC3, for the purposes of this illustration. In real applications it would be necessary to use a much larger number of design levels and make sure that all design levels in the final design level map meet the threshold. It was also checked to see if a higher level of resistance than the one found for life safety would be more economically viable, based on the results in Figure 3.5, and if so, this was selected as the selected design level. Figure 3.5 presents the optimized design level distribution considering both life safety and cost effectiveness.

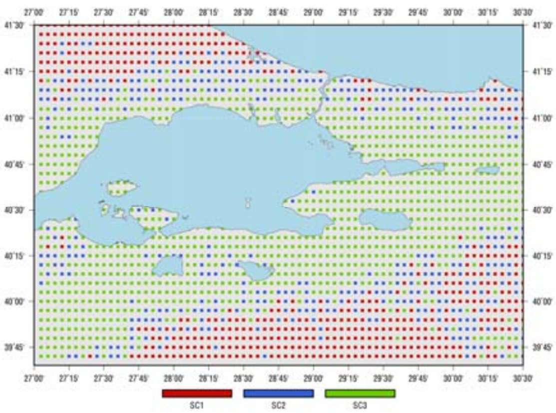

Figure 3.5 Optimized seismic design level distribution considering life safety and cost-effectiveness

As can be seen from Figure 3.5, when considering human losses, the design level map is more conservative, with design level SC3 being required in 53% of the locations while the design SC2 was sufficient in 21% of the sites and the SC1 only fulfilled the requirement in 26% of the sites. This optimized building distribution was used to compute the aggregate cost-benefit factor which was found to be 2253 units. Although this value is slightly greater than the one obtained when considering the optimized distribution based on cost-benefit analysis alone, it is nevertheless important to note that it leads to a lower cost-benefit factor than that found considering any of the three design levels used alone throughout the area, as it is based on an optimal positioning of different levels of seismic design.

4. CONCLUSIONS

The procedures outlined in this paper represent a rather radical departure from the current format of seismic design codes. The philosophy behind the procedure proposed herein is that the experts in engineering seismology and structural earthquake engineering charged with drafting the code should do more of the work outside the code, making the best use of their hazards models and their analytical tools, and only pass on to the engineer in the design office the outcome of these calculations. An important qualification on the procedure outlined herein is that it corresponds to the minimum level of earthquake resistance. In the terminology of current seismic design codes, this would correspond to buildings defined by normal occupancy or low importance factors, i.e. mainly dwellings. For higher occupancy buildings (e.g. schools), hazardous facilities or essential buildings (e.g. hospitals, fire stations), the design levels would clearly need to be increased within each zone. These higher design levels could be obtained in a similar way, primarily by the application of exclusion criteria whereby the buildings in these categories would not be permitted to exceed limit state LS1 or LS2 for a given return period. This could alternatively, or additionally, be specified in terms of a minimum return period at which any such buildings would be expected to exceed one of these lower limit states.

The fact that the proposed procedures distil the hazard, exposure, vulnerability and cost components of the risk equation into a simple zonation map and a suite of design levels specified by combinations of stiffness/ductility, does not mean that it is necessarily desirable to remove information regarding the seismic hazard from the code. The proposed approach of specifying minimum levels of stiffness and ductility is likely to be applicable only to standard building types. Many engineers using the code will have a very good appreciation of the concepts and procedures of seismic design and the code should enable them to carry out checks and enhancements based on expected ground motions and appropriate dynamic analysis of the structure. The question then arises of how these motions should be defined without making recourse to the arbitrary selection of return periods. Alternative formats for presenting earthquake actions in seismic design codes are discussed by Bommer and Pinho (2006) and a thorough description of this issue is provided in Weatherill *et al.* (2010), but it would be feasible to adopt the current approach of defining spectra through one or more mapped parameters, if however these parameters were obtained through disaggregation of the modelled losses that underlay the calibration of the design levels specified. Clearly, an important change from current code formats for compatibility with the framework proposed herein is to move from force-based design using acceleration spectra to displacement-based approaches.

ACKNOWLEDGEMENTS
The authors would like to acknowledge the support provided by the European Commission through the financing of the FP7 research programme SHARE, under the framework of which this work has been partially funded.

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