Cost–benefit analysis of conventional and seismic isolated R/C buildings

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Abstract

The main subject of this work is the cost–benefit evaluation of base isolated versus conventional R/C buildings. By applying well established principles of performance-based earthquake resistant design, using fragility curves obtained from bibliographic sources, integrating a probability density function representing the seismic intensity at the site, and performing approximate non-linear structural analysis, a methodology for investigating the life-cycle costs of an actual three-story R/C building is presented. Through this case study, the use of base isolation is demonstrated to be effective in reducing the life-cycle costs. In addition, by investigation of modified structural systems, it is shown that fewer restrictions are imposed on the architectural design of the building.

Keywords: Performance-based earthquake engineering; Base isolation; Life-cycle costs; Probability density function; Fragility curves; Dynamic analysis; Approximate methods

1. Introduction

The two key issues that must be addressed at the early stages of a design process of a building are the technical and financial feasibility studies. Often ignored, the replacement cost of the non-structural elements and contents of the building damaged by even moderate but frequent tremors, may be too large [1]. Seismic isolation, i.e the earthquake resistant design strategy in which the structure is attached to the ground via flexible in horizontal direction devices-isolators that remain stiff under service loads (winds and moderate earthquake induced ground motions) is a design strategy that reduces *both* interstory drifts and floor accelerations, that induce damage to the building elements. In this work a cost–benefit evaluation of a typical low-rise building situated at a high seismicity region concerning both conventional and isolation strategies, is presented.

2. Performance-based earthquake design

Performance, as a measure of the amount of damage of a facility and the impact of damage to the society after an earthquake, is the main concern of this study. National earthquake resistant design codes usually prescribe performance levels: 'The probability of collapse should be small enough and matched with adequate level of strength after an earthquake with return period of 475 years', but also: 'damage to the structural system for the frequent small earthquakes should be minimized'. This level of earthquake intensity may be correlated to a return period of 50 years in which elastic response is required.

With q denoting the reduction factor of seismic acceleration demand, that accounts for the non-linear response of the real structure during a severe earthquake. Applying this concept for the prescribed performance levels we obtain [2]:

$$\frac{A_{475}^{eff}}{q} \geq A_{50}^{eff} \tag{1}$$

where A_{475}^{eff}, A_{50}^{eff} are the effective peak ground accelerations (PGA) of the ground motion with return periods $T_R = 475$ and $T_R = 50$ years, respectively. Applying Eq. (1) for $A_{475}^{eff} = 0.36\text{g}$ and $A_{50}^{eff} = 0.21\text{g}$, which corresponds to a high seismicity region, we have $q_{max} = 2.00$.

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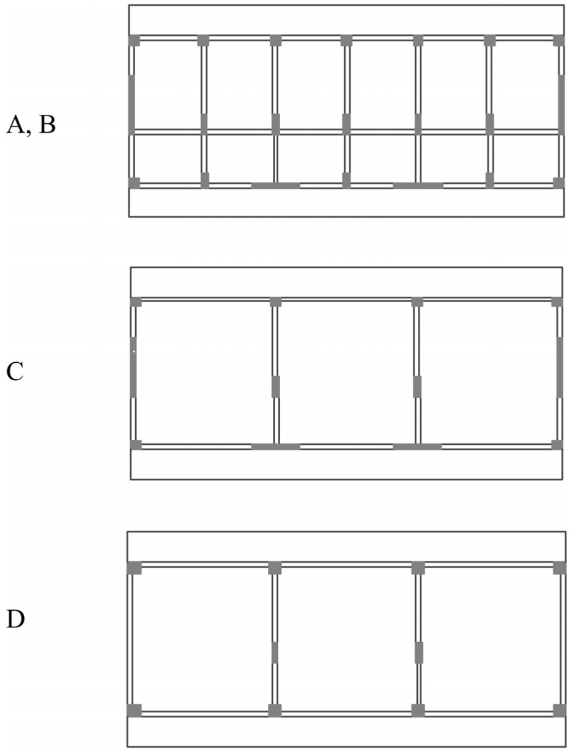

Fig. 1. Structural layouts of buildings A, B, C and D (10.90×21.90 m).

3. Life-cycle cost analysis

The cost of isolation is always an important consideration and usually this is the first question asked by engineers, contractors or owners considering isolation. There are four principal cost factors that may influence the decision making process:

1. The initial cost of construction.
2. Repair costs of damage after earthquakes.
3. Disruption costs due to building and facilities damage.
4. Annual earthquake insurance premium.

In this work only the first two parameters are taken into account, mainly because the others are highly dependent on the specific use of the building and its occupants.

A three-story R/C building with a typical plan dimensions of 21.90 m by 10.90 m is considered. The building has a basement and the lateral force resisting system consists of structural walls and moment frames, as shown in Fig. 1. The conventional building is denoted as 'A' and the isolated one as 'B'.

Lead–rubber bearings (LRBs) are used for isolation, which consist of a laminated elastomeric bearing with a lead insert. They are modeled by a bilinear model based on the elastic stiffness K_1, and the post-yield stiffness K_2. The effective stiffness K_{eff}, defined as the secant slope of the peak-to-peak values in a hysteresis loop at a target displacement D, is given by

$$K_{eff} = K_2 + \frac{Q}{D}, D > D_y \tag{2}$$

where Q is the characteristic strength estimated from the hysteresis loops for the elastomeric bearings and D_y is the yield displacement. It has been found that to a good approximation the total shearing force carried by the lead–rubber bearing F(LRB), is given by

$$F = \tau(\mathrm{Pb}) \cdot A(\mathrm{Pb}) + K_b(\mathrm{r}) \cdot D \tag{3}$$

where the shear stress at which the lead yields $\tau(\mathrm{Pb}) = 10.5\,\mathrm{MPa}$, A(Pb) is the cross sectional area of the lead core, K_b(r) is the stiffness of the rubber in a horizontal plane, and D is the relative displacement at the top of the bearing. The approximate horizontal stiffness K_b is given by

$$K_b = \frac{G \cdot A}{t_r} \tag{4}$$

where G is the shear modulus of rubber and t_r is the total rubber thickness.

The process of recovery of mechanical properties after and during plastic deformation is rapid via the interrelated processes of recovery, recrystallisation and grain growth. With a developed spreadsheet code the bearing stiffness, damping, buckling safety factors, and design displacement are estimated on two-level seismic hazards DBE and MCE (475- and 1000-year return period respectively) [3]. Two modified structural layouts of the same building, denoted C and D, are further investigated in order to establish the ability of the isolated buildings to perform satisfactorily even with less vertical structural elements [1]. The regulations of Eurocodes EC2 and EC8 are applied using ETABS program [4] and the steel reinforcement weight is evaluated as shown in Fig. 2.

The seismically isolated configurations suffer less damage within their design life due to both reduced interstory drifts and floor accelerations. In order to evaluate the total damage costs generated from earthquakes we utilize three main parameters:

1. The seismicity of the site.
2. The damage level of each element related to the induced drifts and accelerations.
3. The construction cost of each element.

To quantify the seismicity of the site a probability density function [5] is derived from seismological data:

$$P = 0.0036 * I^{-2.0371} \tag{5}$$

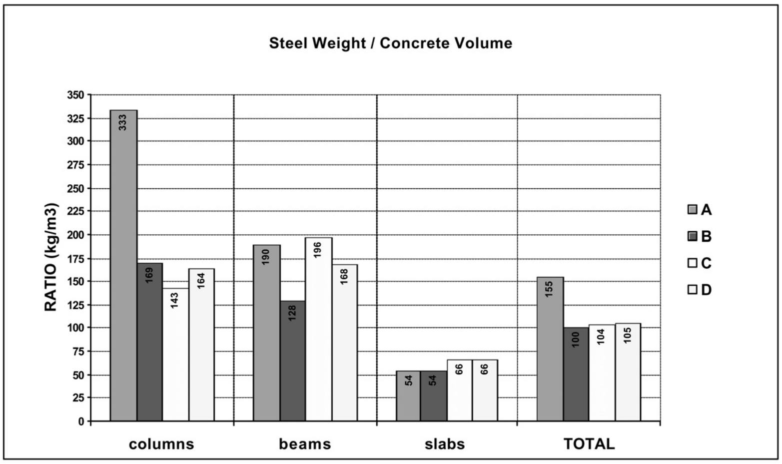

Fig. 2. Ratio of steel weight (kg) per concrete volume (m^3) of the four structural configurations.

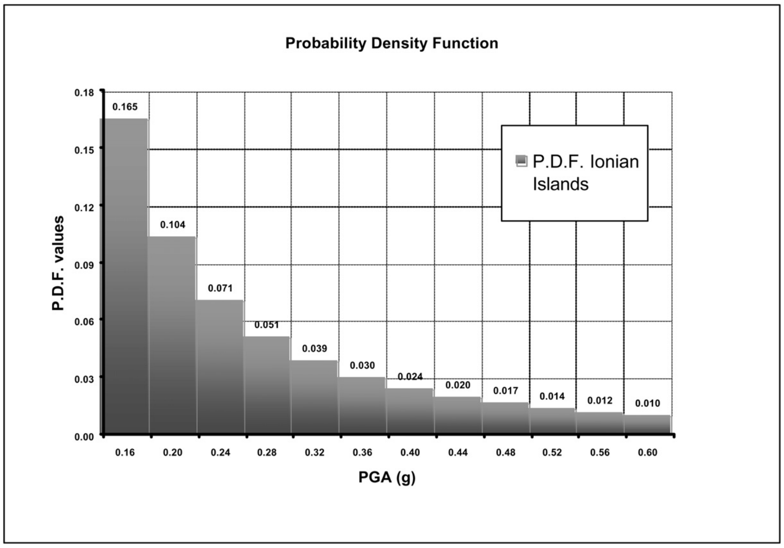

Fig. 3. Discretized probability density function.

where P is the function value and I is the peak ground acceleration. The continuous function is discretized at steps of 0.04g assuming that no damage occurs at PGA below 0.16g and that PGA $=$ 0.60g is the upper bound [1,5], as shown in Fig. 3.

The damage ratios identified by Ferrito [6] can be used as a guide to the potential losses that may occur in the buildings. The advantage of Ferrito's methodology is that it can be applied on both conventional and isolated configurations. Applying the previous suggestions, a mathematical formula is derived:

$$\tilde{C} = \sum_i \sum_j r_j K_j g_j(R_i) P(I_i) \Delta I_i \tag{6}$$

where:

- $P(I_i)$ is the value of the probability density function referred to seismic intensity (PGA) I_i;
- ΔI_i is the discrete step equal to 0.04g;
- $g_j(R_i)$ is the damage ratio of the element j at the induced interstory drift or floor acceleration R_i at each level of PGA I_i;
- K_j is the construction cost of the building element j as a percentage of the total construction cost assuming that the construction cost of the conventional building designed according to EC8 is $C_{EC8} = 1$; and
- r_j is the repair factor of the element j.

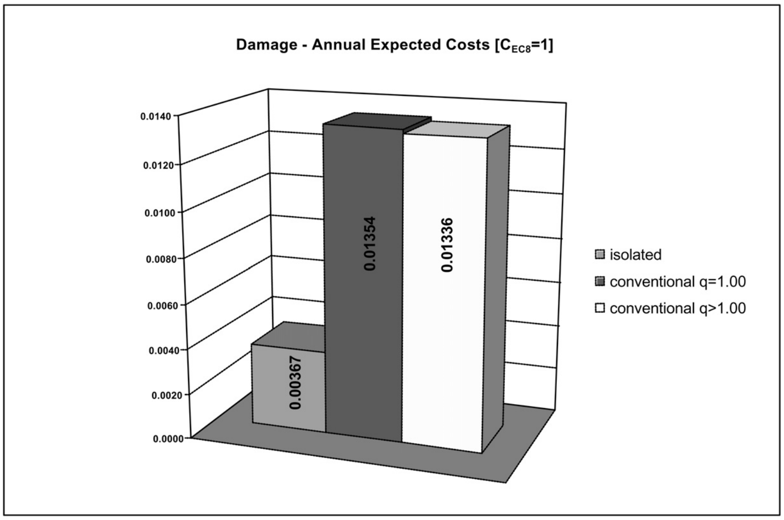

Fig. 4. Annual expected costs of damage.

The results $\tilde{C}$ are expressed as annual cost expectation assuming the design life of the building equal to 50 years (Fig. 4).

Because of the inherent uncertainties in predicting earthquake ground motions, and taking into account the relative accuracy of the multipliers in Eq. (6), an approximate analysis [1] is used to evaluate the non-linear response (R_i) of the conventional building, by performing time-consuming though more accurate non-linear time history analysis. Another reason that such an approximate analysis is used is that, in order to develop an easy-to-apply estimating methodology, relative simplicity is required. The procedure makes use of the fact that at various levels of earthquake intensity the non-linear response of the building is estimated by the previously mentioned q factor. Assuming that at levels of PGA below 0.16g no damage occurs, thus $q = 1.00$, and at the design PGA $= 0.36$g we use $q = 2.00$, the q-factor varies linearly at each step of discretization of the probability density function. Because of recent field observations indicating elastic behavior even when PGA is well above the design value, the calculations are repeated assuming $q = 1.00$ at each step. By applying the definition of the q-factor, interstory drifts are multiplied whereas floor accelerations are divided by q. Thus, the upper and lower bound of interstory drifts and floor accelerations are obtained.

4. Conclusion and findings

If a serviceability limit state is required for a 50-year return period, not only will this limit state control the design, but also innovative design approaches should be used to *economically* satisfy the performance objectives. By applying the proposed methodology, it is shown that the use of base isolation reduces significantly the life-cycle costs of buildings situated at high seismicity regions while at the same time fewer restrictions are imposed on the architectural design.

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