COMPARATIVE ECONOMIC ASSESSMENT OF R/C BUILDINGS WITH INNOVATIVE SEISMIC PROTECTION SYSTEMS

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ABSTRACT :

Knowledge of base-isolation and energy-dissipation is broadly developed all over the world but the number of applications is everywhere limited. The extra-cost associated to the insertion of the protection tools always represents a strong restraint to the application of new technologies. The direct cost comparison induce a wrong evaluation, because it does not take into account for the differences in the performance levels characterising the conventional and enhanced solutions. Comparative analyses of the probabilistic characterization of the seismic response of conventional and advanced variants of the same structural scheme, in terms of story drifts, plastic hinges' rotations, floor accelerations, allow to demonstrate the effectiveness of the innovative protection technologies.

KEYWORDS: Base isolation, energy dissipation, economic assessment, r/c buildings

1 INTRODUCTION

The cost-benefit effectiveness of a protection technique is a fundamental issue for its adoption. The most advanced protection techniques, like base isolation and energy dissipation, generally result in a building more expensive than that designed according to the conventional earthquake resistant principles, due to the cost of the devices to be inserted and of the structural modifications required for their insertion. Only in a reduced number of cases the savings obtained in the main structures and foundations lead to a cost saving for the total building. The illustrated evaluation, based on the only construction costs, has slowed down and continues to slow down the application of these techniques, but it is wrong, because it does not account for the differences in performance of the conventional and enhanced projects. The goal of the conventional design is protecting the human lives without accounting for the structural consequences, on the contrary the advanced protection techniques allow, not only to safe human lives, but also to control both the structural and non structural damage, limiting or even avoiding it.
With the aim of establishing a correct cost-benefit evaluation, procedures are required for carrying out accurate performance-based analyses accounting for a more general concept of the consequences provoked by a seismic attack, that is casualties, rehabilitation costs, availability of the building. Other parameters can be introduced, once the methodology is defined, i.e. indirect consequences correlated to the occupancy of the interior spaces. The performance-based consequence evaluation is the theme of a research work that is currently being carried out within a line of the large DPC funded Italian national research project called ReLUIS, concerning the updating of the earthquake engineering with the up-to-date methods and systems.
First of all a procedure should be outlined allowing for the required calculations. Actually, different methods can be used. Deterministic, fully probabilistic and semi-probabilistic methods can be applied for the consequence evaluation, as illustrated in the first part of the paper. A probabilistic approach is suitable considering the probabilistic nature of the external and internal influence parameters, controlling the structure response, that are managed with the statistic analytical tools. The main scattering sources are: the building characteristics, the entity of the undergone damage, the consequences of the damage on the occupants, costs and downtime. Successively, the paper deals with comparative analyses of the probabilistic characterization of the seismic response of conventional and advanced variants of the same structural scheme, in terms of typical EDP's (Engineered Demand Parameters) - story drift, floor acceleration, residual drift - of conventional fixed-base buildings and innovative protected buildings, demonstrating the effectiveness of the last ones.

2 SEISMIC PERFORMANCE COMPARISON

2.1 *Methodology for performance evaluation*

The correct evaluation of a construction performance should account for the consequences related to different intensities of the seismic input expected of the site. Currently, design codes provide for defining performance requirements concerning the structural response corresponding to at least two levels of the conventional seismic input, usually represented by the same elastic response spectrum scaled for different values of the PGA. A more effective representation of the construction performances should consist on a more general concept of "consequences" correlated to the seismic input.
A comprehensive procedure for the evaluation of the "consequences" should be outlined in four steps:
1) definition of the site hazard in terms of intensity of the expected earthquake, a correlation between PGA and return period can be used, or more complex scenarios can be adopted;
2) definition of a vulnerability function correlating the structural demand to the input, depending on the seismic intensity assumed at design level;
3) definition of damage indicators correlated to the demand level;
4) definition of relationships between damage indicators and consequence parameters.

2.2 *Deterministic methodologies*

Since the very beginning of the application of base-isolation technique, the performance-based evaluation was recognized as the only one able to account for their actual cost-benefits relationships. Methods based on deterministic approaches were formulated (Thiel 1986, Parducci & Mezzi 1995) for establishing a correct cost comparison between fixed-base and base-isolated buildings based on their performances.
The application outlined in (Parducci & Mezzi 1995) provides for a four-step procedure aiming at evaluating the global cost of a construction considering the expected repair costs after the earthquake and assuming it as decision variable for the performance evaluation. The first step consists of the definition of the hazard in terms of intensity of the expected earthquake, a correlation matrix between PGA and return period for different seismic areas can be used. The second step consists of the definition of a vulnerability function expressed by a correlation between the PGA and the global ductility demand, assumed as the parameter characterizing the structural response. The third step should provide a correlation between the response and the structural damage, the same global ductility demand is simply assumed as damage indicator. The final step provides for a correlation between the global ductility demand and the loss parameter, CV_A: a relationship between the damage and the repair cost is used.
In that procedure, the total expected value of the consequence parameter, CV, is computed, for the life duration of the construction, as

$$CV = \int CV_A(R) \cdot dp_R = \int CV_A(R) \cdot \left(\frac{dp_R}{dR} \right) dR \tag{2.1}$$

where R is the return period of the event, dp_R is the associated probability.
The integral (2.1) shall be extended from a lower limit, R_0, that is the return period of that intensity not producing any damage and that can be derived once chosen the value of the demand parameter corresponding to the absence of damage, to a upper limit, R_1, for which the ultimate performance limit hypothesized for the building is reached. This ultimate condition coincides with the complete loss of the building and is characterized by predefined demand level and consequence value, CV_1, corresponding to the cost of substitution. Therefore, calling N the years of the economic life of the construction, the total expected value of the consequence parameter becomes

$$CV = \int_{R_0}^{R_1} \left[CV_A(R) \cdot N \cdot \left(\frac{1}{R} \right)^2 \left(1 - \frac{1}{R} \right)^{N-1} \right] dR + CV_1 \left[1 - \left(1 - \frac{1}{R} \right)^N \right] \tag{2.2}$$

2.3 Probabilistic methodologies

The parameters involved in the performance evaluation - the Intensity Measure of the input, *IM*, the Engineered Demand Parameters, *EDP*, the Damage Measure, *DM*, the Consequence Variable, *CV* - in reality, are not deterministic, but are characterized by statistical distribution due to the uncertainness of the influence variables. Under this assumption guidelines are being currently developed (ATC58, 2005; FEMA445, 2006) providing for a full probabilistic approach in performance evaluation. A consequence is evaluated in terms of probability of exceedance, $\lambda(CV)$, of a predefined level as

$$\lambda(CV)=\iiint G\langle CV \mid DM\rangle dG\langle DM \mid EDP\rangle dG\langle EDP \mid IM\rangle d\lambda\langle IM\rangle \tag{2.3}$$

where each correlation between intensity and demand (EDP|IM), damage and demand (DM|EDP), consequence and damage (CV|DM), is defined through a probabilistic relationship.

2.4 Semi-probabilistic methodologies

A full probabilistic methodology require a big effort for evaluating the consequences in a performance-base design, because of the difficulties in managing the probabilistic characterizations of all the parameters and correlations. Taking into account that the statistical characterization of the parameters is generally stable for classes of homogeneous structural problems, characteristic values can be defined for predefined non exceedance level, as shown in (D'Ambridi & Mezzi, 2005; D'Ambridi & Mezzi, 2008) for typical EDP's.
At each correlation step of the performance evaluation, characteristic values of the parameters can be adopted: $EDP_k(\alpha_{EDP}, IM_k)$, $DM_k(\alpha_{DM}, EDP_k)$, $CV_k(\alpha_{CV}, DM_k)$. Each value is characterized by its non exceeding probability: α_{EDP}, α_{DM}, α_{CV}, that can be chosen for obtaining suitable values of the non exceeding probability of the consequence parameter. This can be estimated, for the considered interval of time, i.e. life duration, as

$$\lambda(CV_k)=\left[1-F_{CV}(CV_k)\right]\cdot\left[1-F_{DM}(DM_k)\right]\cdot\left[1-F_{EDP}(EDP_k)\right]\cdot\left[1-F_{IM}(IM_k)\right] \tag{2.3}$$

where F_{CV}, F_{DM}, F_{EDP}, F_{IM} indicate the CDF (Cumulative Distribution Function) of the PDF (Probability Density Function) of the random variables *CV*, *DM*, *EDP*, *IM*, respectively. Figure 1 illustrates the procedure.

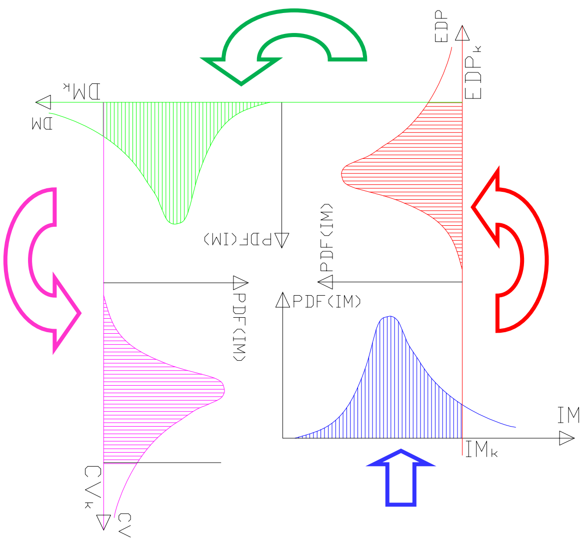

Figure 1 Use of characteristics values of the correlation parameters

3 EFFECT OF ADVANCED PROTECTION SYSTEMS

The factors, that control the probabilistic evaluation of the consequence, depend on different aspects involved in the building process of the construction: conceiving, location, design, construction, use. In particular:

- $d\lambda\langle IM\rangle$ depends on the hazard of the site (*hazard control factor*);
- $dG\langle EDP/IM\rangle$ depends on the design choices (*design control factor*);
- $dG\langle DM/EDP\rangle$ depends on the quality of the construction (*quality control factor*);
- $dG\langle CV/DM\rangle$ depends on the social and economic conditions (*social-economic control factor*).

It is evident that the only parameter that can be influenced by the behavior of the structural system and that, therefore, can be controlled at the design level is the $dG\langle EDP/IM\rangle$, the *design control factor*. On the contrary, it can be assumed that the other factors are similar for different structural systems.

The design decisions play a role through the influence of the *design control factor* in the global probabilistic evaluation. Calling EDP_{DM}, the EDP level for which a consequent damage can be observed, the optimum situation would consist on having a PDF of $\langle EDP/IM\rangle$ such that the $P(EDP>EDP_{DM})\cong 0$ for all the values of IM. Indeed, if a structural system is able to give a PDF of $\langle EDP/IM\rangle$ characterized in such a way, it results in an integral protection, giving values of $\lambda(CV)$, probability to overcame a reference level of consequence, very close to zero, whichever are the PDF of $\langle DM/EDP\rangle$ and $\langle CV/DM\rangle$.

It will be shown, in the paper that structures equipped with special protection system, like base isolation, allow for a practical integral protection with respect to the fixed-base ones.

4 SAMPLE STRUCTURES

The evaluations reported in this paper have been carried out in provision of their application within a probabilistically controlled performance-base methodology. They concern the probabilistic relationship EDP/IM.

The sample structure used for the evaluations is of the 2-bay 4-story plane frame shown in Figure 2, where the main dimensions are reported. Two fixed base variants were analyzed, one (HD) designed in high ductility class and one (LD) designed in low ductility class. Moreover, one base-isolated variant (BI) and one variant with dissipating bracing (ED) were considered.

All the variants are designed, according to the Italian seismic codes, assuming the following data: high seismicity zone, characterized by a PGA of 0.35 g, soil profile type B (medium stiff soil) with an amplification coefficient S=1.25. The dimensions of the members' sections are kept constant for all the variants: 300x500m for the beams, 400x400 mm for the columns. Due to the reduced number of floors the members' sections do not vary along the height. The variants differ in the rebar percentage, computed according to the required strength.

The following loads have been considered. Dead loads of the structural members: 25 kN/m^3. Unitary dead and permanent loads at floors: $g_{k,s}$=6.0 KN/m^2. Unitary live loads: $q_{k,s}$=2,0 KN/m^2 at the intermediate floors, $q_{k,s}$=1,5 KN/m^2 at roof level. Loads have been applied assuming a 5 m long floor span. Structural factors q=5.85 and q=4.095 have been used for designing the HD and LD variants, respectively.

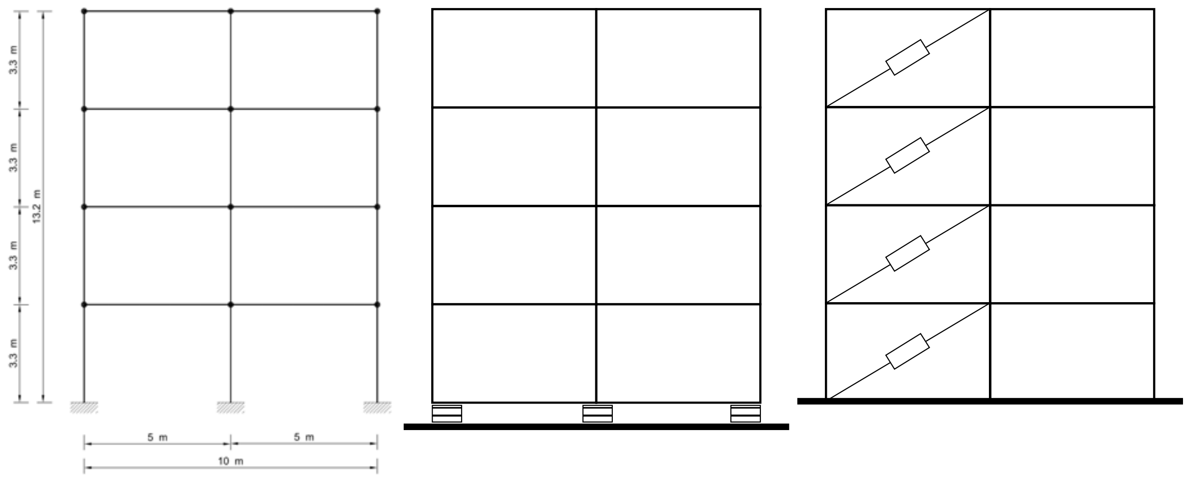

Figure 2 Variants of the sample structures

4.1 Isolated frame

The base-isolated variant (Figure 2b) has the same configuration and sections' dimensions of the fixed base one. It includes a ground floor level above the isolation layer and differs in the rebar percentage of the r.c. members.
The isolating system consists of HDRB devices characterized by the following parameters:
- secant stiffness Ke = 400/kN/m;
- damping, as ratio to the critical value, $\xi_e = 0.10$.

For the dynamic analyses a non linear modelling of the devices is assumed represented by an hysteretic behavior characterized by the following parameters:
- maximum displacement and force $d_2 = 300$ mm, $F_2 = 120$ kN;
- initial elastic stiffness $k_1 = F_1/d_1 = 1600$ kN/m;
- post-elastic stiffness $k_2 = (F_2-F_1)/(d_2-d_1) = 335$ kN/m;
- elastic to secant stiffness ratio $k_1/k_e = 4$;
- yielding displacement and force $d_1 = 0.016$ m, $F_1 = 25$;
- energy dissipated in a load cycle $W_d = 2\,\pi\,\xi_e\,F_2\,d_2$

The period of the fundamental mode results T_1 = 2,42 s, with a mass participating ratio equal 0,999.
According to the Italian guidelines the elastic response spectrum was reduced by the coefficient $\eta = 0.8165$ in the range over $T = 0.8 \times T_1$, accounting for the isolation system damping. The design of the elevation r/c members was carried out adopting a structural factor $q = 1.15 \times \alpha_u/\alpha_1$, that, assuming $\alpha_u/\alpha_1 = 1.3$ for a multi-bay multi-floor frame, results $q = 1.495$.

4.2 Dissipated frame

The dissipated model is characterized by diagonal dissipating braces inserted in the first bay at all the stories, according to the scheme reported in Figure 2c. The variant has the same configuration and sections' dimensions of the fixed base one, differing only in the rebar percentage of the r.c. members.
The mechanical characteristics of the devices have been defined on the basis of empirical evaluations and maintained constant at all the stories. An elastic-perfectly-plastic behaviour of the devices is assumed, hypothesizing to simulate EP devices like the BRD (Buckling Restrained Devices) or highly non linear viscous dampers. The yielding force of the dissipating device is assumed equal 211 kN. The adopted solution simulates a dissipated system with an equivalent percent damping estimated around 15%. The period of the fundamental mode results T_1 = 0,51 s with a mass participating ratio equal 0,859. The design of the members was carried out using a spectrum computed for a structural factor $q = 4.095$ and a coefficient $\eta = 0.707$ corresponding to the estimated damping.
Actually, the adopted configuration reproduces a slight dissipation system. Higher dissipation levels can be hypothesized with the available commercial devices, so obtaining larger effectiveness of the protection. In the present step of the research only the described dissipation level was considered.

4.3 Dynamic non linear analyses

Non linear time history analyses are carried out, on all the variants, using seven recorded spectrum-fitting accelerograms selected from the European Strongmotion Database (www.isesd.cv.ic.ac.uk). The accelerograms reported in (Iervolino et al., 2006) for 2D analyses, selected according to the criteria there reported for matching the elastic spectrum for soils class B, have been used.
The main EDP's characterizing the structural response are considered: top displacement, story drift ratios, plastic hinges' rotations, floor accelerations, spectral floor accelerations. The probabilistic characterization of these parameters is carried out assuming a log-normal PDF according to the assumptions reported in the literature (ATC58, 2005; D'Ambrisi and Mezzi; 2005).
The adopted plasticity model is based on the rising up of plastic hinges at the member ends when the forces in the corresponding sections overwhelm their elastic strength limits. For guaranteeing the continuity of the interval of definition of the rotations, an assumption has been adopted for representing the elastic state of the hinges, lacking in plastic rotation: in the elastic range a pseudo-rotation value is defined by multiplying the rotation at the elastic limit by the ratio of the actual moment to the yielding moment.

5 ENGINEERING DEMAND PARAMETERS

5.1 Top displacement and base shear

The first monitored demand parameters are the top displacement and base shear. Even if it is not directly correlated with the structural performance in terms of consequences, it can be considered a general indicator characterizing the global structure behaviour. If correlated with the base shear it gives a representation of the lateral capacity of the whole structure, as it is commonly done in the non linear static analysis.
The left diagram of Figure 3 reports the fragility curves of the top displacement, expressed as ratio to the total building height, for the four considered variants of the sample r/c frame. The different variants show very different scattering with minimum values got from the BI solution. If looking at the fragility curves, it can be seen that the base isolation has the practical certainty (100% of non exceeding probability) to maintain the top displacement below a global drift ratio 0.004 (48 mm) for which damage can be excluded. The ED has 25% of exceeding probability associated to the global drift 0.005: maybe this value could reduce adopting a more effective dissipation. On the contrary, the fixed-base solutions, both HD and LD, have a 85% probability of exceeding 0.005 and about 40% to overcome a global drift ratio of 0.01. The base shear with non exceeding probability of 50% of the BI variant is less than half that of fixed-base variants, while that of the ED variant is more than 20% larger.

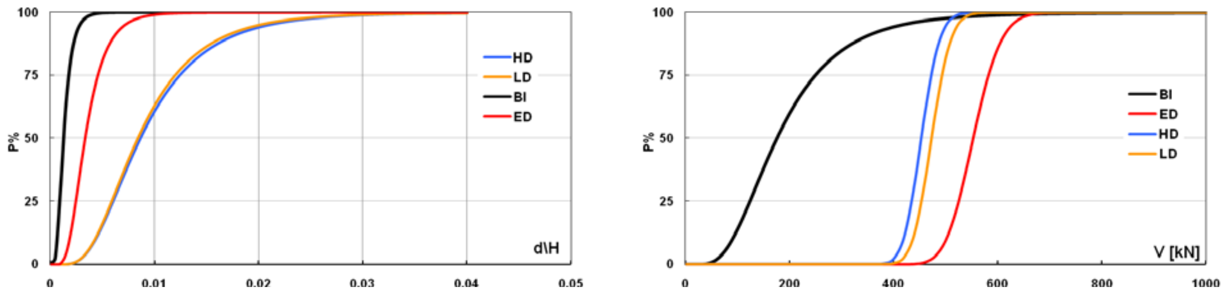

Figure 3 Fragility curves of the top displacement and base shear of the four variants

5.2 Story drift ratios

The story drift ratio is the most used EDP for describing the structural performance because it is correlated with both structural (Yong et al., 2005; Botta & Mezzi, 2008) and non structural damage. Current guidelines (FEMA357, 2000) express the structure performances in terms of this EDP, when assessing the damage status.
Figure 4 shows the fragility curves of the drift ratio at the first and second stories for the four variants. The non exceeding probability of the story drift ratio 0.005 - conventionally considered corresponding to the absence of damage - is 100% for BI variant, therefore it performs within the status of complete protection. The non exceeding probability becomes 45% for the ED variant, that anyhow shows an 85% with respect to the drift 0.01, the threshold over which significant damage shall be expected. HD and LD variants do not differ and show a large probability to overcome the threshold of 0.01: 70% at the first story, 65% at the second story. Moreover they show a probability of about 30% to undergo severe damage, corresponding to drift ratios larger than 0.02.

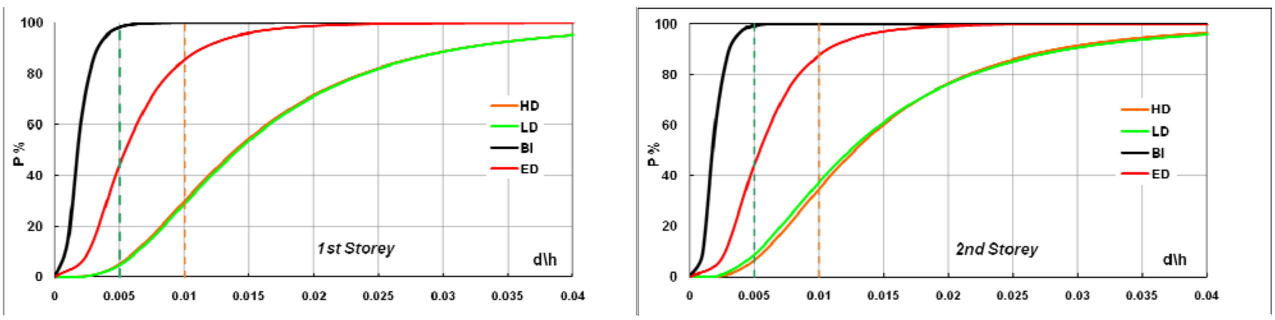

Figure 4 Fragility curves of the story drift ratio at 1st storey (left) and 2nd storey (right)

5.3 Plastic hinges' rotations

Within the adopted plasticity model, the local EDP's of the structural element can be represented by the rotations of the plastic hinges. Figure 5 shows, as an example, the fragility curves of the plastic rotations of the bottom and top hinges of the 1st-story left column for the four variants; in each diagram are also evidenced, for each variant, with the same colour, the levels corresponding to the initial (dashed lines) and ultimate (dash-dotted lines) damage. Similarly, Figure 6 shows the fragility curves of the plastic rotations of the left and right hinges of the 1st-floor left beam for the four variants.
The left diagram of Figure 5 shows that in BI variant the non exceeding probability of the no damage rotation (0.007 rad, corresponding, for the columns, to the Immediate Occupancy limit) is 100%, confirming the status of integral protection got with base isolation. The non exceeding probability in the fixed-base variant is near to 10%. Therefore both HD and LD variants performs in the range of controlled damage, but they show non negligible probability value (more then 10%) of overwhelming the ultimate damage level. The ED variant, even not performing as well as the BI one, fully remains within the limits of the controlled damage, always reminding that only a slight dissipation has been simulated. The diagram on the right evidences that no plastic rotation are developed at the top section of the columns, because at the 1st floor level the plasticization interests only the beams, according to the principles of the Capacity Design, under which the sample models have been designed. For the beams, which plasticize at the both ends, the diagrams of Figure 6 evidence behaviours similar to those described for the columns, and the same comments can be extended.

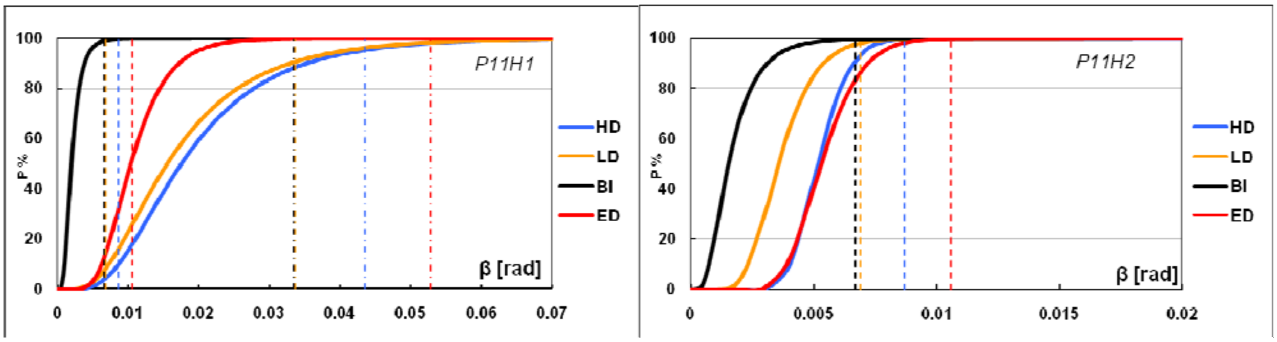

Figure 5 Fragility curves of the plastic rotations of the bottom and top hinges of the 1st-story external column

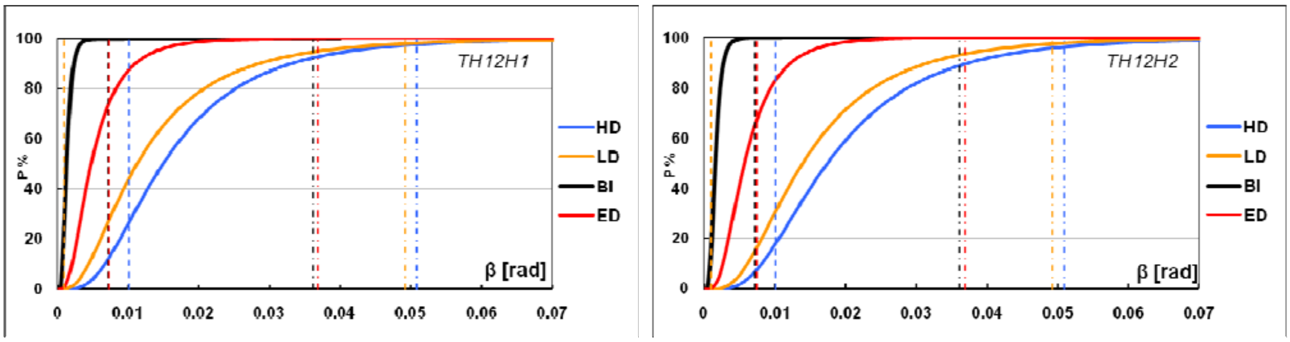

Figure 6 Fragility curves of the plastic hinge rotations of beams in the four variants

5.4 Floor accelerations

The fragility curves of the peak floor acceleration, a significant parameter aimed at accounting for the performance of non structural elements, and of spectral floor acceleration were derived. Figure 7 shows the fragility curves of the peak floor acceleration at 1st floor and 4th floor. The diagrams evidence a more uniform response, in terms of both value and scattering, of the base isolated variant along the height. Contrarily to the expectation, the isolated solution does not give a significant reduction of the response acceleration if compared to the high ductility fixed base solution where the large plasticization of the base members gives an isolation-like effect. For the sake of brevity, no considerations are done on spectral floor accelerations.

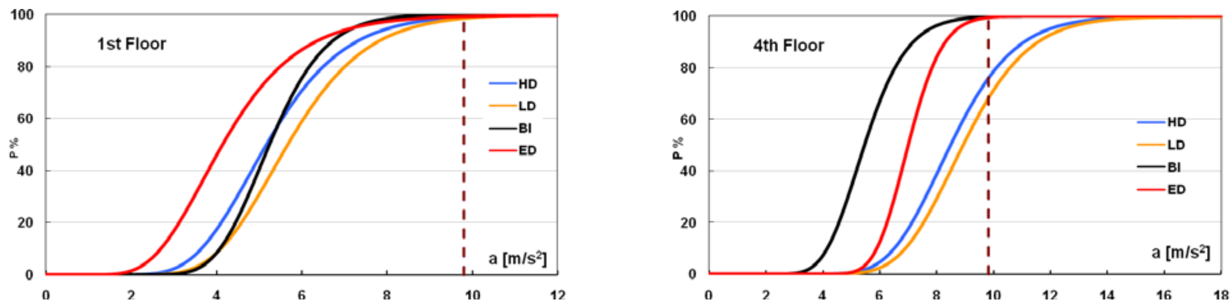

Figure 7 Fragility curves of the peak floor acceleration at 1st floor and 4th floor.

6 CONCLUSIONS

Procedures are summarized aimed at defining a correct comparison between conventional fixed-base and base-isolated or dissipating braced buildings. The methods requires the probabilistic characterization of the correlations among seismic input, structural demand, member damage, and consequent losses. Comparative analyses of the probabilistic characterization of the main EDP's have been carried out with reference to four variants of the same sample building. EDP's fragility curves show that base isolated buildings have null exceedance probability with respect to critical EDP limits for which damage can start. This condition basically controls the performance of the protected buildings reducing the relevance of the successive probabilistic steps.

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REFERENCES

ATC58 (2007). Guidelines for Seismic Performance Assessment of Buildings 35%draft. Applied Technology Council, Redwood City, CA, USA.
Botta, M., Mezzi, M. (2008). Probabilistic correlation of damage and seismic demand in R/C structures. Int. fib symposium 2008, Amsterdam, Nederland.
D'Ambrisi, A., Mezzi, M. (2005). A Probabilistic Approach for Estimating the Seismic Response of EP SDOF Systems. Earthquake Eng. & Struct. Dynamics. 34(14), 1737-1753.
D'Ambrisi, A., Mezzi, M. (2008). Assessment of a probabilistic method for evaluating residual response parameters of ep structural systems. 14WCEE, Beijing.
FEMA357 (2000). Global topics report on the prestandard and commentary for the seismic rehabilitation of buildings. Federal Emergency Management Agency, Washington, D.C., USA.
FEMA445 (2006). Next-Generation Performance-Based Seismic Design Guidelines. Federal Emergency Management Agency, Washington, D.C., USA.
Iervolino, I., Maddaloni, G., Cosenza, E. (2006). Accelerogrammi Naturali Compatibili con le Specifiche dell’OPCM 3274 per l’Analisi Non Lineare delle Strutture. 6° Congresso CTE. Parma, Italy. (in italian)
Parducci, A., Mezzi, M. (1995). Economics in Seismic Isolation Options Expected Benefits Versus Construction Costs. PCEE, Melbourne, Australia.
Thiel, C.C. (1986). Life Cycle Cost Consideration in Structural System Selection. Applied Technology Council ATC-17. San Francisco, California.
Yong, L., Xiaoming, G., Jiong, G. (2005). Probabilistic Drift Limits and Performance Evaluation of Reinforced Concrete Columns. J. Struct. Div. ASCE, 131(6), 966-978.