PERFORMANCE ASSESSMENT OF ISOLATED STRUCTURES WITH ADDED DAMPING

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ABSTRACT

Seismic isolation and use of damper technologies seen a variety of applications in the design of structures to mitigate seismic risk.Viscous Damper (VD) is used to improve the performance of a base isolation system and decreasing displacement demand. When the damping in the bearings is low, use of viscous damper is the most effective for reducing the structuralresponse. Lack of clear-cut method for selecting design parameters and the combination of these two systems and decision on how to reduce structural damage caused reluctance of clients to use these systems. Performance-based design provides a useful framework for developing the relationships among the characteristics of the ground motion, base-isolated structure and viscous damperand to evaluate the ability of design approaches to achieve targeted performance goals. The purpose of this paper is to develop methods for considering performance parameters like: mean annual frequency of limit states, mean annual frequency of collapse and collapse margin ratio (CMR) in performance assessment of base isolated structures with viscous damper. Studies showed that base isolated structure with viscous damperhave better seismic performance comparably to base isolated structures.

INTRODUCTION

In the last 20 years, seismic isolation and dampers have been very effective in the design of structures to mitigate Seismic risk (Morgan and Mahin, 2011). The function of the isolators is distinguishing between the main frequency of the structure and the powerful earthquake frequency domain. Isolated Structure tends to respond like rigid mass in a way that majority deformations can take place in the flexible layer of the isolator. The isolator drastically reduces the main frequency of the structure and subsequently lowers the acceleration of the floors. While this flexible layer protects the building from destruction, it undergoes a relatively large displacement demand. In order to lessen this displacement demand; supplementary energy dissipation mechanisms are proposed. If dynamic vibration absorber used with flexible isolators, it can control the reaction of the structure by limiting displacements and forces.

With high-velocity pulses andhigh displacement demands, many near field (NF) situations require impractical isolator bearing dimensions and designs. Consequently, supplemental damping is needed to reduce the horizontal displacement demands, otherwise structural integrity could potentially be jeopardized (Providakis, 2008). The combined isolation system of HDR or LRB with viscous dampers seems to work well in the NF regions. Unfortunately, this combined system does not perform desirably in moderate or strong FF events due to the secondary forces produced by the dampers and their complex coupling effects (Providakis, 2008), as well as the higher modes effects.

To address the problem stated above, the efficiency of providing different LRB systems for actual RC buildings, in combination with supplemental dampers, was investigated by many researchers. The response of this combined isolation action as well as the superstructure behaviour seems to be effective for NF ground motions (Alhamaydeh etal., 2013).

Lack of clear-cut method for selecting design parameters and the combination of these two systems and decision on how to reduce structural damage caused reluctance of clients to use these systems. Performance-based design provides a useful framework for developing the relationships among the characteristics of the ground motion, base-isolated structure and viscous damper and to evaluate the ability of design approaches to achieve targeted performance goals.

Performance based earthquake engineering (PBEE)

Performance-based earthquake engineering seeks to improve seismic risk decision-making through assessment and design methods that have a strong scientific basis and that express options in terms that enable stakeholders to make informed decisions. A key feature is the definition of performance metrics that are relevant to decision making for seismic risk mitigation. The methodology needs to be underpinned by a consistent procedure that characterizes the important seismic hazard and engineering aspects of the problem, and that relates these quantitatively to the defined performance metrics (Moehle and Deierlein, 2004).

The mathematical formulation for evaluating decision variables and providing decision support to the owner/user, considering uncertainties inherent in all parts of the process, is provided by the PEER framework equation expressed as follows:

$$\int [DV] = \iiint G(DV \mid DM) dG(DM \mid EDP) .dG(EDP \mid IM) d(IM).d(EDP).d(DM) \tag{1}$$

Where termsλ[X], G (X|Y), DV, DM, EDP and IM represent, the Mean Annual Frequency (MAF) of exceeding X, the Complementary Cumulative Distribution Function (CCDF) of X conditioned on Y, Decision Variable, Damage Measure, Engineering Demand Parameter and Intensity Measure, respectively.

PBEE methodology consists of four successive analyses: hazard analysis, structural analysis, damage analysis, and loss analysis. The methodology focuses on the probabilistic calculation of meaningful system performance measures to facility stakeholders by considering the above four analyses in an integrated manner where uncertainties are explicitly considered in all four analyses (Gunay and Mosalam, 2012). Each analysis is described separately in the following subsections.

Hazard Analysis

Intensity measures are quantities that describe the magnitude (M) of ground motion characteristics that significantly affect the upstream variables of the performance assessment approach. In the context of Eq (1), this implies an evaluation of the Mean Annual Frequency (MAF) of IMs through seismic hazard analysis. The 5%-damped first-mode spectral acceleration $S_a(T_1\ ,5\%)$ is indeed the best choice from simplicity and accuracy standpoints (Zareian and Krawinkler, 2009).

Structural Analysis

The amount of demand induced in structure and nonstructural component is represented by term G(EDP|IM) in Eq (1). Once identified, they can be computed from different procedures such as, incremental dynamic analysis (IDA) procedure (Vamvatsikos and Cornell, 2002), which accounts for the record-to record uncertainty that attributed to the aleatory nature of earthquake hazard. In this procedure, the soil–foundation–structure system is subjected to a ground motion whose intensity is incremented after each inelastic dynamic analysis. The result is a curve that shows the EDP plotted against the IM used to control the increment of the ground motion. IDAs can be carried out for a sufficiently large number of ground motions to perform statistical evaluation of the results.

Damage and loss analysis

According to uncertainty in damage extent at a specific response level term (DM|EDP) is used. To estimate the damage in structural elements, a relationship between relevant EDPs and different levels of damage, referred to here as Damage Measures (DM) are required. The selected damage states reflected the necessary measures of fixing a member and then turning it back to be perfect. Fragility functions are used for damage analysis that are are acquired on the basis of experimental tests, evaluations conducted from the analysis, experts' opinions, and the experience from previous earthquakes.

Because of uncertainties in the number of people and facilities and their location (when an earthquake occurs) term (DV|DM) is used. After an earthquake, the repair cost will not be the only "loss" suffered by building stakeholder and a variety of factors can affect the consequences of a decision as a Decision Variable (DV). Assuredly considering as many factors as possible in the decision-making process will cause the most precise results. For decisions regarding the effects of seismic events on the buildings, these consequences include mortality as well as direct and indirect economic losses. The challenge lies in incorporating these factors and their consequences, given a particular course of action, into a measurable quantity that can be used to define performance measures. Addressing this challenge is crucial as any decision is ultimately judged on the consequences of its outcome.

In order to evaluate the isolator parameters and viscous damper in seismic performance, a suitable set of demand parameters should be selected. The damage caused by the earthquake is usually attributed to interstory drifts. Damage states of the structural parts are calculated by a response vector containing maximum interstory drift that is used in IDA.

One of the important points in PBEE is the approximation of the structure performance under seismic loads, specially determining the mean annual frequency (MAF) of seismic demands or the determined limit state capacity. Applying this method on the structure performance has different steps. In first step, the appropriate non-linear model of structure and a set of accelerograms are selected. Then each of these accelerograms is scaled with a suitable method and then the non-linear dynamic analysis is conducted and at last the results are analyzed and then structure IDA curves can be developed (damage measure versus intensity measure).

This method is applied for each accelerograms and the results of the distribution of damage measure for determined intense measure are summarized. Then limit states are defined on the IDA curves and are summarized to acquire the probability of exceedance of a determined limit state. The results in this appropriate can be combined with the hazard curves (PSHA) for calculating the mean annual frequency of certain seismic demands exceedance(Vamvatsikos and Cornell, 2002).

Occurrence probability of limit states

To extract the occurrence probability of limit states from IDA analysis output, fragility curves are used. In order to develop these develop, intensity measure concurrent with the intended limit styles, is ascending sorted on the basis of all records. According to this, occurrence probabilities of limit states are calculated with cumulative density function (CDF) and delineated with IM in fragility curves. Using this curve, probability of the limit styles for each IM level, if the IM value has limited to the specified level, can be calculated.

In order to be able to carry out the performance calculations, limit states on the IDA curves should be defined. Three commonly- used limit states are considered namely Immediate Occupancy (IO), Collapse Prevention (CP) as per FEMA 350 (2000), and global dynamic instability (GI) are chosen. For the models in this study, Immediate Occupancy is defined according to FEMA guidelines. The IO limit-state is defined atθ_{max}=1% for OMF. As for CP point two criteria are to be met. The first one simply involves a local tangent of the IDA curve which is not to be less than 20% of the initial elastic slope. If this local tangent becomes less than 20% of the initial slope, CP is violated. The second criterion is defined atθ_{max}=10% for OMF. Whichever occurs first in IM terms, decides the CP limit state. Finally, GI is encountered once flat-lining of IDA curve is reached and any increase in the IM results in practically infinite DM response.

Mean annual frequency of limit states

Recent performance based guidelines propose a scheme for assessing the mean annual frequency (MAF) or annual probability of exceeding a limit state for a given structure at a designated site. This scheme de-convolves the assessment by introducing DM and IM in an integral form as the equation below (Asgarian et al., 2010):

$$\lambda[\mathrm{LS}] = \iint_{\mathrm{DM,IM}} \mathrm{G}[\mathrm{LS}|DM]\,|dG[DM|IM]|\,|d\lambda[IM]| \qquad (2)$$

In this equation,λ[LS]is the mean annual frequency of exceeding the limit state LS and G[LS/DM] denotes the probability of exceeding the LS given the value of DM and finally G[DM/IM] denotes the probability of exceeding each value of DM given the value of IM. By using probabilistic seismic hazard analysis, one could explicitly obtain the termλ[LS] which is in other words the hazard curve of a given site. This value ofλ[LS] is directly obtained if one tries to evaluate each term explicitly and solves for the results using any numerical integration method. Two performance objectives (i.e. IO and CP) are chosen to be the target objective.

Collapse Margin Ratio and the annual frequency of collapse

The Seismic Design Procedures Group suggests that a conditional probability of collapse of 10% given that the building is subjected to Maximum Considered Earthquake shaking is an acceptable goal. For most sites in the, Maximum Considered Earthquake (MCE) shaking has an annual frequency of exceedance of 1 in 2,500 or 4x10-4 per year. When integrated with the hazard curve, this typically results in an annual Frequency of collapse (ATC-58, 2009).

As defined in Eq(3), the collapse margin ratio, CMR, is the ratio of the median 5%-damped spectral acceleration of the collapse level ground motions, $\widehat{S}_{CT}$, to the 5% damped spectral acceleration of the MCE ground motions, S_{MT} at the fundamental period of the seismic-force-resisting system:

$$CMR = \frac{\widehat{S}_{CT}}{S_{MT}} \qquad (3)$$

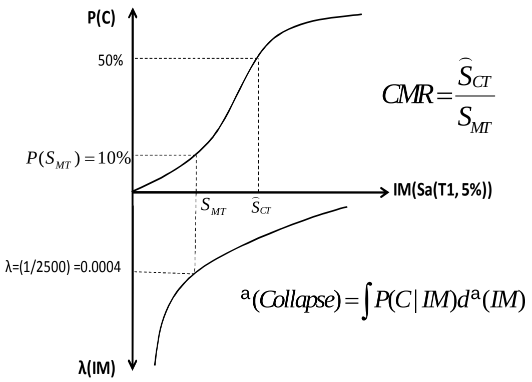

Figure 1. Collapse Margin Ratio (CMR) and Mean Annual Frequency of Collapse (λ(Collapse))

CASE STUDY STRUCTURE

Case Study was performed on the 5-story perimeter steel resisting frame located in Tehran city. ASCE 7-10 used for seismic design requirements for seismically isolated structures. The software Sap2000 is used for primary design. Frames contain three-bay 5 meters span and story height of 3.5 meters.According to FEMA P695 (2009), a set of 28 pairs of ground motions recorded at sites less than 10 km from fault rupture, referred to as the "Near-Field" record set is considered for analysis.Linear viscous damper with α=1&C_d=300 (kN.s/cm) andlow damping rubber (LDR)with ξ=10%are used for this study.

Modeling of Frames using OpenSess

Considering the existing symmetries and regardless of the probable lack of symmetry in seismic loads on the structure, and that only two moment frames will resist on each direction, only one of the frames will be analyzed by OpenSess in 2D model.In order to consider the effect of the gravity load applied on the inner frames on the demand of exterior moment frames, leaning columns is used. These columns are rigid parts that are connected by the pinned rigid beams to the frame. In order to calculate the effect of the P-delta

caused by loading of inner frames, weight of these spans (due to symmetry of half of them) has been forced on the end ties of the rigid beams. When side movement takes place in the structure, these forces make secondary moment in the moment frame.
Nonlinear beam-column elements with concentrated plastic hinges in two ends, connected by an elastic element, are adopted for modeling the frames. The nonlinear behavior in plastic hinges is modeled implementing rotational springs (with stiffness and strength deterioration). Cyclic moment-rotation of steel beams and columns are represented by Lignos and Krawinkler(2007) which focuses on development of a steel component database that can serve as the basis for validation and improvement of analytical models that explicitly model deterioration in structural steel components and can be used in collapse assessment of steel moment resisting frames. In addition, in order to consider the cyclic deterioration, the modified model suggested by Ibarra et al. (2005) have been used. The definition of plastic hinges has been performed using Joint2D-5spr element in OpenSees with panel zone modeled with Gupta et al. (1999) model as depicted in Figure 2. Column base uplift, as expectable in isolated structures, has been considered through definition of zero-tension spring elements at the column-foundation interface.The elastomeric bearing element is used for modelling of base isolation which uses the mechanical properties of elastomeric bearing as input parameters to describe the force-deformation relationships. A user can assign any material model available in the OpenSees material library in the vertical (axial) direction.Also viscous dampers has been modelled with Maxwell model ThisViscousDamper material simulates the hysteretic response of viscous dampers.

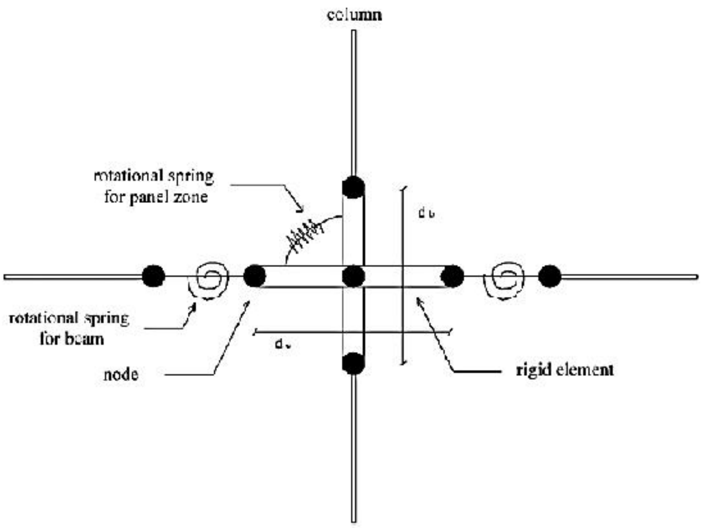

Figure 2. Model of panel zone (Gupta et al. (1999))

IDA Results

In Figure 3, the fragility curve of56accelerogramsfor CP is shown for all 3 structures with lognormal distribution. Corresponding values of S_a with occurrence probabilities of CP and IO are shown in table 1. It can be inferred from this table that, structure's S_a capacity increases, as the time period of the structure increases.

The median values of maximum interstory drift, among from 56accelerograms for all 3 structures, is shown in Figure 4. As denoted in Figure 4, the isolator has increased the drift in comparison to the primary frame. Also VD has increased the lower floors drift of isolated frame, but it acts more efficiently in high floors and reduces the drift. VD has reduced 15% of isolated frame's drift in the last floor.

Table 1.Summarized Sa for each limit-state

S_a	Primary Frame		Isolated Frame		Isolated Frame with Added Damping	
	IO	CP	IO	CP	IO	CP
16%	0.21	0.67	0.32	1.23	0.42	1.34
50%	0.32	1.12	0.46	1.85	0.54	2.09
84%	0.41	1.62	0.56	2.94	0.65	3.12

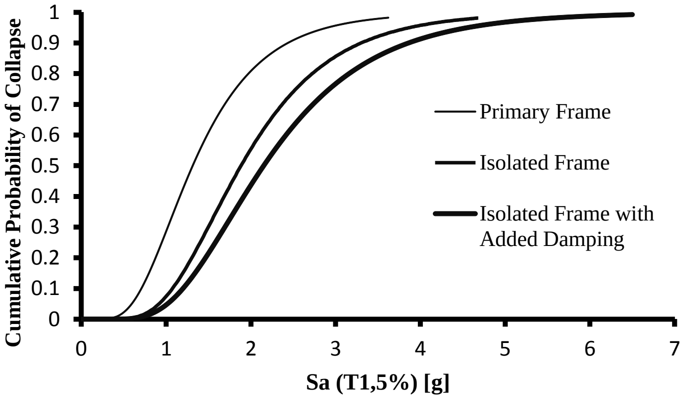

Figure 3.Collapse fragility curves for frames with lognormal distribution

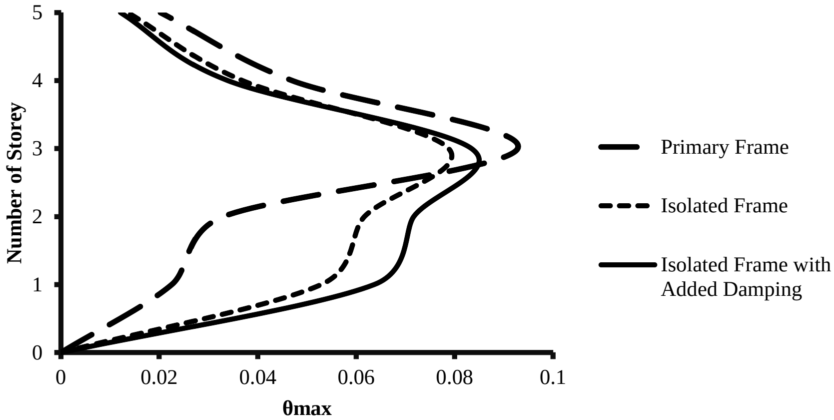

Figure 4.Comparison of interstory drift of frames for 50% probability of collapse

Comparison of performance parameters

The seismic hazard parameters used in this part of the study, is extracted by MahdaviAdeli et al. (2011). In the mentioned study, according to different reduction terms, the vibration risk is measured at different parts of Tehran, and using the calculated-average of seismic hazard values, the uniform hazard map of Tehran is drawn. By using seismic hazard curves and fragility curves the MAF value of IO and CP, for all 3 structures, is shown in table 3. These values, as quantities that reflect the whole probable capacity of structures with uncertainty caused by the earthquake, are very useful. We can use these values as criteria for the safety of structures in comparison with other structures, or use them in design regulatory criteria of structures.

CMR and mean annual frequency of collapse for earthquake with the probability of 1% in 2500 years, is shown in table 4. It is clear that structure with isolator and VD, shows more satisfying results than structure with isolator, regarding seismic demands, and has more appropriate CMR and mean annual frequency of collapse. Also, in comparison with the primary structure, a considerable increase (increase of 2.13 times) in CMR has occurred in isolated structure.

The MAF of collapse and CMR are not applicable for structures with different floors and heights, because the seismic hazard depends on time period of structure, and in this case study, considering that the building is mid-rise steel structure, isolators increase time period of structure and reduces seismic risk. It is possible that in high-rise structures, variation of MAF value of specified limit state, due to increase in time period can be more effective than reduction of seismic hazard.

Table 3:MAF value of IO and CP, for all 3 structures

All Cases	**Primary Frame**		**Isolated Frame**		**Isolated Frame with Added Damping**	
	IO	CP	IO	CP	IO	CP
MAF of Exceedance	5.44E-4	1.03E-4	1.2E-4	1.19E-5	6.16E-5	6.67E-6

Table 4:Collapse Margin Ratio (CMR) and Mean Annual Frequency of Collapse for MCE

All Cases	Primary Frame			Isolated Frame			Isolated Frame with Added Damping		
	Sa_{MCE}	λ_{MCE}	CMR	Sa_{MCE}	λ_{MCE}	CMR	Sa_{MCE}	λ_{MCE}	CMR
Results	0.41g	1.38E-5	2.45	0.289g	2.65E-8	5.22	0.226g	2.94E-9	7.36

CONCLUSIONS

Performance assessment presented in this paper, provides a good framework for evaluating structures containing isolator and viscous damper, to achieve the targeted performance goals. Hence, a framework was set and using performance parameters like: spectral acceleration for operational levels, mean frequency of exceedance of limit states, annual frequency of collapse, collapse margin ratio, performance evaluation for structures containing isolators and dampers was conducted. The offered method facilitates operational comparison among different structural systems.

Studies showed that base isolated structure with added damping have better seismic performance comparably to base isolated structures regarding seismic demand and the CMR and mean annual frequency of collapse. Also there has been a considerable increase of CMR in isolated structure against the primary structure.

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