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Damage potential reduction of optimally passive-controlled nonlinear structures

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ABSTRACT

The use of energy dissipation devices as an earthquake resistant system is an increasing area of practical application worldwide. They are used as seismic protection systems in order to enhance structural performance by increasing inherent damping, raising safety, controlling deformations and balancing asymmetric conditions among others, thus reducing ideally the overall damage potential of the structures. Few codes have been developed for the analysis and design of these structures and the existing ones are still evolving. A key parameter that requires special attention is the added damping – reduced shear base design relationship due to, in a philosophical sense, seismic protected structures should assure the assumed performance enhancement. A comprehensive study of the added damping – structural strength relationship is conducted through three stages. The first stage is focused on nonlinear single degree of freedom systems, the second is extended to nonlinear 2-DOF systems in order to consider the torsional condition, and lastly a real eight story frame is analyzed. Structural performance is estimated via fragility curves constructed through the Incremental Dynamic Analysis. Damage is estimated through the Park and Ang index which is composed of two main parameters, the first related to damage caused by ductility demand, the second related to structural hysteretic dissipated energy. An ensemble of 42 Chilean accelerograms recorded at the M_w 8.8 2010 Maule Earthquake scaled to fifteen increasing intensities is considered. An optimal plan and height distribution of damping capacities is considered for the 2-DOF system and the eight story frame respectively. The Simplified Sequential Search Algorithm is the optimization technique implemented. Besides fragility curves, an added damping – strength capacity chart is presented as a decision-making tool. Structural strength reductions should be carefully considered in order to assure an effective damage potential reduction, especially when dealing with asymmetric structures.

1. Introduction

The concept of performance-based earthquake engineering is implicitly part of early modern design codes which are oriented to achieve the life safety limit state under a design level earthquake intensity. Nevertheless major earthquakes such as those of Northridge (1994), Kobe (1995) and more recent ones such as Maule (2010) and Tohoku (2011) have shown that a single performance-based seismic design does not allow to consider seismic risk as a design and decision tool. Moreover, regardless of the limited building losses during these earthquakes, building owners expected significantly higher performance levels specially when the repair cost of nonstructural elements, equipment, and interrupted operations represented larger expenses than the value of the structure itself. Life safety limit state uniquely assures that a structure may not collapse when subjected to a design-level earthquake even if it may not be functional again [1].

Raising the seismic performance of a structure while keeping costs reasonably is an arduous task which becomes more challenging being aware that increasing strength may not necessarily enhance safety, nor reduce damage [2]. Asymmetric structures are particularly vulnerable to the seismic action [3] due to the concentration of deformation at some resisting planes focusing damage in a small number of elements.

The use of Energy Dissipation Devices (EDD) is an advisable solution in order to enhance structural performance. The use of metallic or friction dampers placed at strategic self-amplifying locations such as wall core lintels may provide an economic solution. Nevertheless the seismic response of structures equipped with EDD is sensitive to their spatial distribution. Several methods have been proposed for the optimal distribution of EDD, most of

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them based on the linear response of the main structure and assuming linear viscous dampers [4–6]. Despite these methodologies are highly efficient, they are neither simple, nor practical for their application. A preceding investigation [7] concerned about the optimal location of EDD showed that the solution of a sequential methodology such as the Simplified Sequential Search Algorithm (SSSA) [8,9] converges to the optimal solution of rigorous optimization techniques such as the Min–Max Algorithm (MMA). This optimal solution not only reduces but also equalizes drift deformation at every story and at the peripheral frames achieving the so called drift and torsional balance [10]. Even more, it shows that the optimal location of EDD is lightly influenced by the inelastic behavior of the main structure nor the nonlinear response of the devices.

This investigation assesses the performance enhancement (damage reduction) of asymmetric structures optimally passive controlled through fragility analyses.

2. Fragility analyses

The quantification of the potential damage of a structure subjected to a strong ground motion is a complex task mostly performed in terms of a probability statement [11].

Fragility analysis is actually a tool of performance based engineering. It consists in developing a series of fragility curves which typically denote the probability of exceeding a limit state as function of the earthquake intensity (hazard). These curves can be developed through diverse approaches [12,13]. In this investigation fragility curves are constructed through the Incremental Dynamic Analysis (IDA) [14,15] which is a probabilistic procedure.

IDA is a parametric analysis method that estimates thoroughly structural performance under seismic loading. It allows picturing the complete range of structural behavior: form elasticity to yielding and finally collapse. It has also been adopted by the US Federal Emergency Management Agency guidelines [16] as the state of the art method to determine the global collapse capacity. This approach consists in performing nonlinear time-history dynamic analyses for an ensemble of earthquake ground motion records, each scaled to increasing intensity levels. A performance index is assessed and retained for each record scaled to each intensity and then compared with a reference limit state such as those defined by FEMA 356 [17]: Operational (OP), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), which are widely used in other studies [18]. Fragility curves are constructed as a cumulative probability function by counting the number of ground motion records causing the performance index to exceed the selected limit state.

2.1. Record scaling and ground motion selection

Ground motion record selection and scaling techniques are by themselves an important issue. They are still evolving and there is still debate as to which method is the most appropriate [19]. So, as considered by the Pacific Earthquake Engineering Research Center (PEER), we have selected a large number of ground motion accelerograms to obtain statistically robust results, which are less dependent on the specific choice of records. Given the limited availability of recorded ground motions within relatively narrow magnitude and distance ranges appropriate to a given site conditions, these constraints are relaxed in favor of other parameters that are better predictors of the nonlinear response. The records selected belong to both horizontal directions of the 21 strongest accelerograms recorded at the M_w 8.8 2010 Maule, Chile Earthquake (Table 1).

The record scaling technique consists in developing 15 spectrums, each representing a recurrence interval (earthquake intensity) ranging from 2% to 90% probability of exceedance in 100 years. Then, each accelerogram (that can be interpreted as a function in the time domain) is converted thorough the Fourier transform to the frequency domain. Each series of the function in the frequency domain can be scaled to match the intensity of the spectrum. Then, the signal is reverted and the result is a spectrum – compatible accelerogram. The seed spectrum (Fig. 1) corresponds to the design spectrum of the Chilean Isolation Code [20] for firm soil which is a Newmark-Hall type spectrum compatible with the scaling techniques described by FEMA [17] and the IBC [21]. The reference intensities defined by FEMA corresponding to the frequent earthquake (return period of 43 years), occasional earth-

Table 1
Dynamic characteristics of the 21 strongest accelerograms recorded at the Maule Earthquake. Chile, 2010.

Location	Maximum acceleration (g)			LIF-L* (s)	LIF-T* (s)	Total record length (s)
	Longitudinal	Transverse	Vertical			
Copiapó	0.03	0.016	0.008	60.0	57.7	70.0
Vallenar	0.02	0.019	0.01	57.2	59.7	69.0
Papudo	0.295	0.421	0.155	44.8	49.9	88.0
Viña del Mar (Marga–Marga)	0.351	0.338	0.261	47.3	47.4	170.0
Viña del Mar (Center)	0.219	0.334	0.186	39.5	47.3	125.0
Valparaiso (Univ. Tec. F.S.M)	0.137	0.304	0.079	31.4	26.4	72.0
Valparaíso (Almendral)	0.224	0.265	0.146	43.3	41.6	102.8
Llolleo	0.319	0.564	0.702	42.2	37.3	124.6
Santiago (Cener)	0.218	0.309	0.182	40.3	38.4	205.0
Santiago (Maipú)	0.561	0.478	0.24	40.5	45.4	167.0
Santiago (Peñalolen)	0.295	0.293	0.28	39.1	48.6	171.0
Santiago (Puente Alto)	0.265	0.263	0.13	42.7	52.8	147.0
Santiago (La Florida)	0.236	0.165	0.13	42.3	44.0	208.0
Matanzas	0.342	0.308	0.234	42.2	40.9	120.4
Hualañe	0.389	0.461	0.39	64.1	59.6	144.0
Curico	0.47	0.409	0.198	54.2	55.3	180.0
Talca	0.477	0.424	0.244	73.3	74.7	148.0
Constitución	0.552	0.64	0.352	59.7	66.5	143.3
Concepción	0.402	0.284	0.398	81.0	84.5	141.6
Angol	0.928	0.681	0.281	50.4	62.3	180.0
Valdivia	0.092	0.138	0.051	50.9	50.6	79.0

*LIP-L = Length of the intense phase – Longitudinal.
*LIP-T = Length of the intense phase – Transversal.