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RESILIENT SEISMIC DESIGN OF TALL COUPLED SHEAR WALL BUILDINGS USING VISCOELASTIC COUPLING DAMPERS

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ABSTRACT: The effectiveness of viscoelastic coupling dampers (VCDs) in improving the seismic performance of a 40-story reinforced concrete (RC) building designed using a non-prescriptive performance-based approach is investigated. Three-dimensional nonlinear finite element models are developed and response time-history analyses are carried out. Significant reductions in all response indicators of the conventional building are achieved when VCDs are used in an outrigger configuration in combination with VCDs replacing coupling beams in the concrete core. In particular, by using VCDs in this configuration, median values of peak inter-story drift ratios are reduced by up to 46% under design level earthquakes (DE) and by up to 39% under maximum considered earthquakes (MCE) relative to the conventional building. More importantly, the conventional building which suffers moderate damage under DE, remains essentially elastic when the VCDs are incorporated. Loss estimation analyses revealed that VCDs can provide up to a 45% reduction in financial losses under the DE and up to 41% reduction under the MCE compared to the conventional building. These results indicate that the VCDs greatly enhance the seismic resilience of the conventional RC building by significantly reducing structural as well as non-structural damage throughout the height of the building and by allowing quicker inspection and repair following DE- and MCE-level events.

1. Introduction

Resilience of tall buildings in moderate- to highly-seismic regions around the world is of increasing concern because of uncertainties associated with their expected performance in the event of a strong earthquake and the consequences of having damage distributed over the height of these buildings requiring repair. According to the current design codes in the US (ASCE 2010), for buildings greater than 48 m in height, special lateral force resisting systems such as a dual system comprised of perimeter moment-resisting frame and a reinforced concrete (RC) core wall should be used. Since having two redundant lateral systems can significantly increase the cost of the project, adversely affects architectural layout, and may not result in a better structural performance, a performance-based approach that allows going beyond the usual code limits is followed and a single robust RC core wall system is used for seismic resistance. In the performance-based design of such buildings, typically damage control or collapse prevention limit-states are targeted under a rare maximum considered earthquake (MCE). Because of the explicit use of nonlinear analysis, and a rigorous peer review process, it is expected that tall buildings will perform better than conventional low- to mid-rise buildings under strong earthquakes. Nonetheless, recent studies have shown that significant structural and non-structural damage and business interruption due to downtime can occur even under design earthquake (DE) for conventional tall RC buildings (MacKay-Lyons et al. 2012; Tipler et al. 2014). Furthermore, life-cycle repair costs of tall buildings under a range of wind and seismic hazards over a life-time can be significant. One of the main factors affecting resilience of conventional tall RC buildings is their dependence on uncertain inherent damping values and the uncertainties related to the inelastic response of RC elements under design- and beyond design-level earthquakes. The ideal performance of a tall building in which the structure remains essentially elastic under DE, is difficult to achieve effectively and economically using conventional methods. Therefore, supplemental damping systems that not only provide a reliable source of energy dissipation but also significantly reduce structural and non-structural damage, are highly desirable. Providing supplemental energy dissipation in tall RC buildings is challenging and most of the available damping systems could be ineffective over a wide range of wind and seismic hazards (Christopoulos and Montgomery 2013). Therefore, for efficient protection of tall RC buildings against wind and seismic vibrations, the viscoelastic coupling damper (VCD) has been recently developed (Montgomery 2011). The VCDs consist of multiple layers of viscoelastic (VE) material sandwiched between alternating steel plates. When VCDs are used at appropriate locations in a tall building (see Fig. 1(a)), the VE material undergoes cyclic shear deformation under wind and earthquake excitations providing an efficient instantaneous energy dissipation mechanism from low to large shear deformations. A ductile fuse mechanism can be introduced in series with the VE material such that the fuse activates for extremely large earthquakes and limits the forces transferred to the concrete walls and prevent tearing of the VE material (Fig. 1(b)). The VCDs are configured to be replaceable and following a major seismic event, they can easily be inspected and replaced if required. More details on VCDs including their full-scale experimental validation can be found elsewhere (Montgomery and Christopoulos 2014).

In the present study, viscoelastic coupling dampers (VCDs) are used to enhance the seismic resilience of a 40-story RC building under multiple seismic hazard levels. The conventional building is designed using a state-of-the-art performance-based methodology with the objectives to remain essentially elastic under service-level earthquake (SLE), suffer moderate damage under DE, and avoid collapse under MCE. Using VCDs, this study is primarily aimed at achieving essentially elastic response of the building under DE. A three-dimensional nonlinear model of the building is developed and response history analyses are carried out under a suite of site-specific ground motions. Finally, loss estimation is carried out to investigate the impact of providing VCDs on financial losses under DE and MCE.

2. Description and Design of the Conventional Building

A 40-story residential RC building designed for Seattle, Washington was investigated. The building also incorporates an additional 5 stories below grade and a 3-story penthouse and mechanical floor at the top (Fig. 2(a)). The typical story height is 3 m and the total height of the building from foundation is 148 m. The primary lateral load resisting system of the building consists of a coupled RC core wall (Fig. 2(b)). The gravity system consists of 200 mm thick post-tensioned RC slabs resting on RC columns. The podium of the building is shared with another 12-story office building, which is not considered in this study because it is separated using seismic joints.

2.1. Seismic Hazard

A detailed site specific seismic hazard study was carried out by Hart Crowser (2014). The seismicity of the site is dominated by the Cascadia Subduction Zone and the Seattle Fault Zone. The building site is located in a sedimentary basin within 5 km of the northern splay of the Seattle Fault Zone. Site-specific response spectra were developed for the risk-targeted maximum considered earthquake (MCE_R) and

SLE. The DE response spectrum was taken as two-thirds of the MCE spectrum. Based on the de-aggregation of the seismic hazard, a ground motion source distribution of four interface subduction and three crustal earthquakes was identified. Ground motion time histories (Table 1) were then selected from historic earthquakes. The crustal earthquakes were rotated to fault-normal (FN) and fault-parallel (FP) components. The ground motion time-histories were scaled such that the average square root of the sum of the squares (SRSS) spectrum of the ground motions does not fall below the MCE_R spectrum for the periods up to 10 s (Fig. 2(c)). Since the site is located within 5 km of an active fault, it was ensured that the average response spectrum of the FN components of the crustal earthquakes also does not fall below the MCE_R spectrum (ASCE 2010).

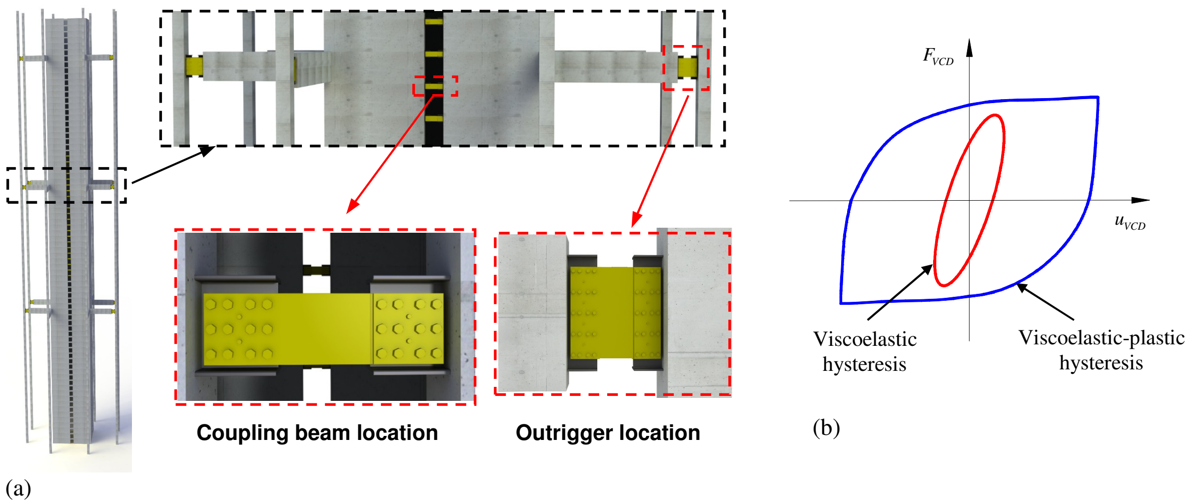

Fig. 1 – (a) Possible VCD locations in a tall building (slabs are not shown for clarity) and (b) *viscoelastic* hysteretic envelopes for wind and moderate earthquake loading and *viscoelastic-plastic* hysteretic envelopes for extreme seismic loading.

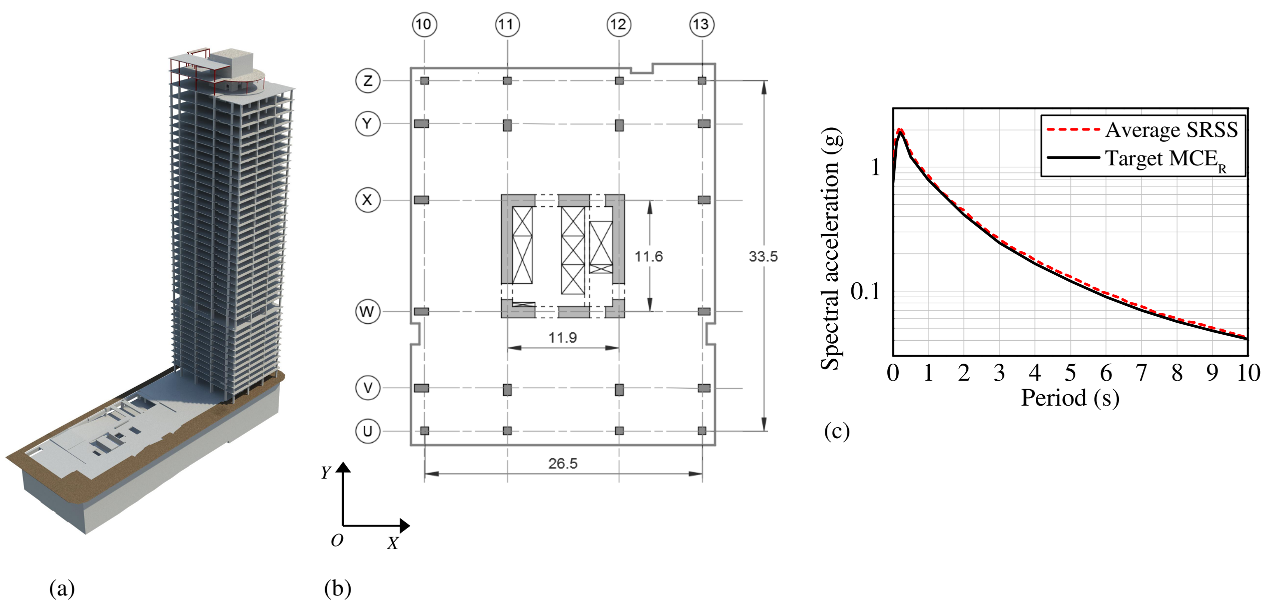

Fig. 2 – (a) Isometric view of the building; (b) typical floor plan (all dimensions are in m); and (c) comparison of the average response spectrum of ground motions scaled to represent MCE with the target spectrum.

Table 1 – Earthquake ground motions considered in this study.

EQ No.	Event	Year	Station	M_w	Closest dist.[a] (km)	Fault mechanism
1	Chile	2010	CCSP	8.8	36.2	Interface subduction
2	Tohoku	2011	Ujie, TCGH12	9.0	103.8	Interface subduction
3	Tohoku	2011	Oyama, TCG012	9.0	119.4	Interface subduction
4	Tokachi-Oki	2003	HDK084	8.3	146.8	Interface subduction
5	Tabas, Iran	1978	Tabas	7.35	2.1	Crustal - reverse
6	Loma Prieta	1989	LGPC	6.93	3.9	Crustal – reverse oblique
7	Loma Prieta	1989	WVC	6.93	9.3	Crustal – reverse oblique

[a]Hypocentral distance for EQ #4. All other distances are closest distance to fault rupture.

2.2. Design Philosophy

The building was designed in accordance with the 2012 International Building Code (ICC 2012) using a performance-based approach. The performance objectives were (i) essentially elastic response under service-level earthquake (SLE), (ii) damage control (moderate damage, but extensive repair may be required) under design earthquake (DE), and (iii) collapse prevention under risk-targeted maximum considered earthquake (MCE_R). Coupling beams and core walls in flexure (see the next section for details) were allowed to undergo nonlinear deformation. All other components including core walls in shear were designed to remain elastic at all hazard levels. For SLE and DE response spectrum analyses were carried out in ETABS (2013)), while for MCE nonlinear response history analyses (RHAs) were carried out in PERFORM-3D (2013)) using the set of 7 ground motions. The next section provides details of the nonlinear model in PERFORM-3D. The acceptance criteria were mainly based on the PEER TBI guidelines (PEER 2010) and LATBSDC guidelines (LATBSDC 2014). Analysis results revealed that the maximum peak inter-story drift ratios under SLE, DE, and MCE were 0.08%, 0.77%, and 1.1% (median of responses from 7 ground motions; the average drift ratio was 1.27%), respectively. This indicates that the building is expected to suffer essentially no damage under SLE ground motions while moderate damage is expected under DE- and MCE-level motions such that extensive repair would be required.

2.3. Description of the Nonlinear Model in PERFORM-3D

A three-dimensional nonlinear finite element model of the building was developed in PERFORM-3D. Expected material properties were used rather than nominal properties. A standard trilinear backbone curve was used for simulating the response of the nonlinear material models and inelastic hinges. Reduced effective stiffness properties were used for elements modeled to remain elastic, to take into account the effects of cracking. The RC core wall was modeled using *shear wall elements*. The axial and in-plane bending response of the shear wall elements was modeled using inelastic fiber cross sections, while shear force-deformation relationship was assumed to be elastic. For modeling of fiber cross sections, the *inelastic 1D concrete material* model with strength loss was considered. The tensile strength of concrete as well as degradation of material properties under cyclic loading was neglected. For reinforcing steel, the *non-buckling inelastic steel material* with no strength loss and cyclic degradation of properties was used. Coupling beams were modeled using elastic beam elements with a nonlinear shear hinge at mid-span. Strength loss as well as cyclic degradation of properties was considered for the inelastic shear hinges (Naish et al. 2009). Horizontal rigid beam embeds with negligible axial and torsional stiffness were used to connect the coupling beam elements to the shear wall elements (Berahman 2013). Floor slabs at and below grade level were modeled using thin elastic shell elements. Floor slabs above grade level were simulated using elastic equivalent *beam element*s extending from each corner of the core wall to the adjacent gravity column. Columns were modeled using elastic *column elements*. The mass of each floor above grade level was lumped at a master node located at the centroid of the floor and all other nodes on the floor were constrained to the master node using a rigid diaphragm constraint. Inherent damping was simulated using Rayleigh damping with a damping ratio of 2.5% targeted for the modes significantly contributing to the response. P-Delta effects were considered for the shear wall and column elements.

3. Performance Enhancement using Viscoelastic Coupling Dampers (VCDs)

3.1. Considered VCD Configurations and Numerical Modeling

Several different configurations of VCDs were evaluated for enhanced seismic performance of the building. In this paper results of two viable configurations that provide significant benefits are presented. In Configuration A, concrete outriggers were introduced at 12 FL and 35 FL and 1.5 m long VCDs were placed between the outrigger and adjacent column (Fig. 4(a)). Note that this configuration can be carefully designed so that the architectural layout of the building is not affected and the occupiable space is not lost. This can be achieved by strategically placing outrigger walls between apartments and by allowing open corridor around the elevators by using coupling beams between the concrete outrigger and the core wall instead of using a solid outrigger wall or by using VCDs between the outrigger and the core wall. In Configuration B, in addition to the damped outriggers from Configuration A, coupling beams at levels 4-12 and 27-35 were replaced by VCDs (Fig. 4(b)). Also, in Configuration B, to avoid a sudden change in the stiffness due to incorporation of VCDs, stiff concrete beams were replaced by W21X182 steel beams which had similar capacity and stiffness compared to the VCDs. In total, 64 and 172 VCDs were used in Configurations A and B, respectively. The VCDs were designed through an optimization process considering practical limitations in terms of the size of the dampers and the existing coupling beams as well as available space. The VCDs in the outriggers can be configured seamlessly within the building such that no architectural space is occupied (Fig. 4(c)). The details of the VCDs replacing the RC coupling beams are shown in Fig. 5(a)-(c).

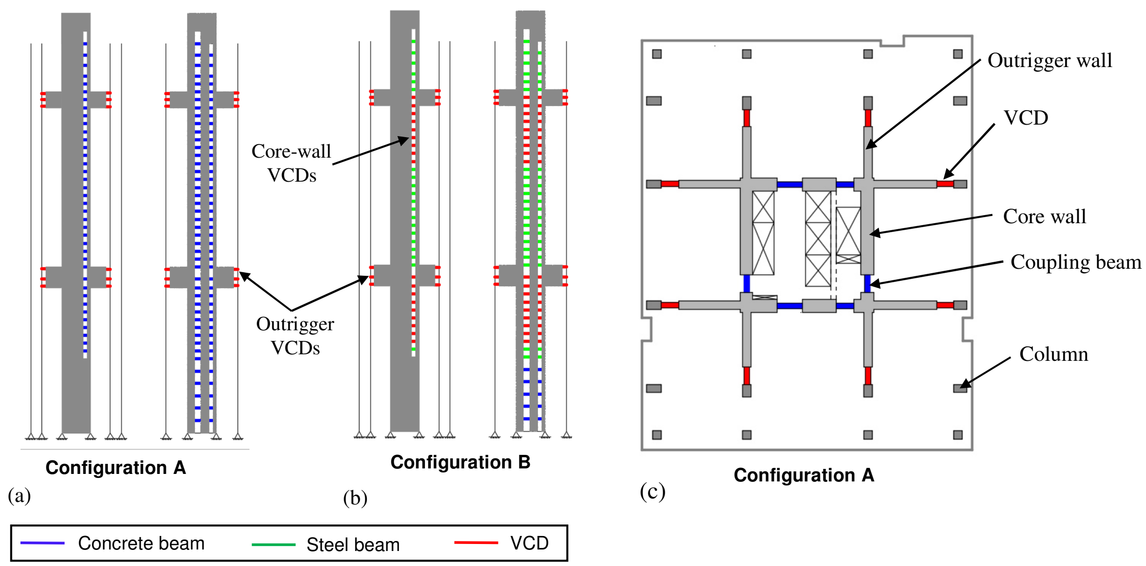

Fig. 4 – (a)-(b) Elevations along grid lines 11 and X (see Fig. 1(b)) for Configurations A and B (elevations along grid lines 12 and W are also similar) and (c) typical outrigger floor plan for Configuration A.

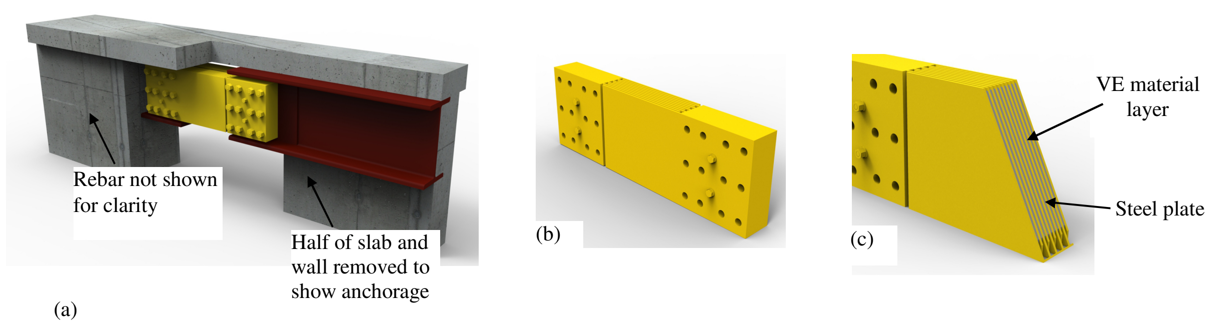

Fig. 5 – (a) Viscoelastic coupling damper (VCD) in the location of a coupling beam; (b) VE damper panel; and (c) cross-section view of the VCD.

Both VCD configurations were incorporated in the PERFORM-3D nonlinear model discussed in Sec. 2.3. The VCDs were modeled using a Generalized Maxwell Model (GMM, proposed by Fan (1998))) to simulate VE material, in series with a spring element to simulate the connection stiffness (Fig. 6(a)). The GMM model adopted in this study consists of one spring in parallel with two Maxwell elements and is capable of capturing the frequency dependence of the VE material response. As shown in Fig. 6(b)-(c), this GMM model has been validated using the results from tests conducted on full scale VCD specimens by Montgomery and Christopoulos (2014)). A reference temperature of 24^0 C was used to calculate VCD parameters, because the VE material properties in the VCDs are not expected to change significantly under seismic loading (Montgomery and Christopoulos 2014). The steel coupling beams were modeled using elastic beam elements with nonlinear shear hinges at mid-span. Nonlinear response history analyses were then carried out using the set of 7 ground motions that are shown in Table 1.

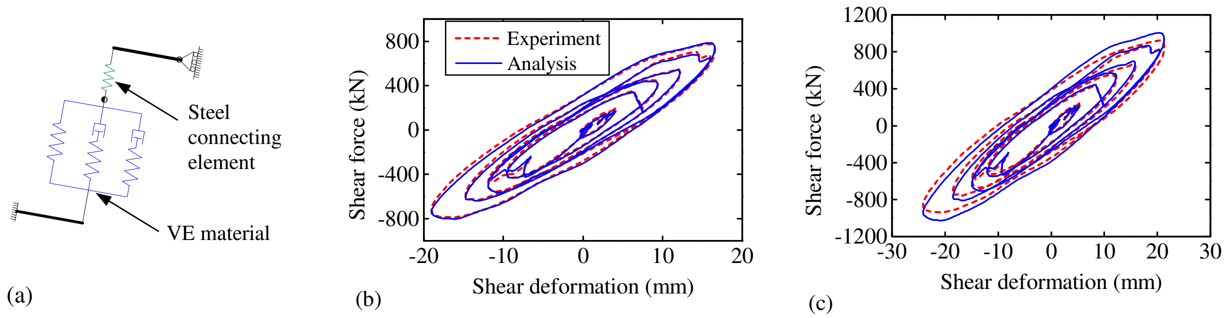

Fig. 6 – **(a) Schematic of the numerical model of the VCD and (b)-(c) validation of the model based on full-scale tests of a VCD in a 50-story building subject to SLE- and DE-level 1992 Landers Earthquake, respectively.**

3.2. Effects on Peak Response Indicators

The building response in *X*- and *Y*-directions is discussed in terms of the peak values of inter-story drift ratio, absolute floor acceleration, coupling beam plastic rotation, core wall shear force ratio, and core wall bending moment ratio, evaluated as median responses obtained from all 7 ground motions. It is noted that although average values are often preferred in practice, median values have been chosen for comparison purposes, as a review of previous studies shows that median values can be less biased by extremely large or small demands imposed by only a few ground motions in a set (Pant et al. 2013; Terzic et al. 2014). The inter-story drift ratios were computed as the maximum of the drift ratios recorded at each of the four corners at each floor of the building. In general, an inter-story drift ratio of less than 0.5% corresponds to an essentially elastic response, while that in the range of 0.5-1.5%, usually corresponds to moderate damage where extensive repair may be required (Elnashai and Di Sarno 2008; PEER/ATC 2010). The floor accelerations were taken at the center of mass at each floor. Coupling beam plastic rotations were taken as the maximum plastic rotation of all beams at each floor in each direction, with zero plastic rotation implying an elastic response. The core wall shear forces were normalized by the total seismic weight and the core wall bending moments were normalized by the total seismic weight times the total height of the building.

Since the building behaves exceptionally well under SLE and suffers essentially no damage, only the results under DE- and MCE-level ground motions are discussed in this paper. Story-wise distribution of peak response indicators under *DE-level motions* are shown in Fig. 7, while the change in maximum values of peak response indicators in comparison with the conventional building is shown in Table 2. It is observed from Fig. 7 that the *conventional building* suffers moderate structural damage as indicated by the maximum inter-story drift ratio of 0.82% (Fig. 7(a)-(b)) and coupling beams undergo inelastic deformations with a maximum plastic rotation of 0.65% (Fig. 7(c)-(d)). Floor accelerations remain less than 0.4*g* (Fig. 7(e)-(f)), which is a threshold at which point damage in acceleration-sensitive non-structural components and contents is expected to increase (PACT 2012; Soroushian et al. 2015). The median core wall base shear was about 9.3% of the total weight of the building and the core wall base bending moment was about 2.3% of the total weight times the total height of the building (Fig. 7(g)-(j)). Thus, although the conventional building's response meets the design objectives, it undergoes significant inelastic deformations and extensive repair would likely be required following a DE-level event. By adding VCDs in *Configuration A*, significant reductions in inter-story drift ratios are achieved throughout the height of the building compared with the conventional building (Fig. 7(a)-(b)). In particular, the maximum inter-story drift ratio in the *Y*-dir. is reduced by 33% (see Table 2). Nonetheless, adding VCDs in this configuration does not reduce the coupling beam rotations and floor accelerations (Fig. 7(c)-(f)). While core wall bending moments show some benefit of adding VCDs in Configuration A, as shown by the reduced base moments in the *X*-direction and nearly identical base moment in the *Y*-direction, core wall shear forces increase throughout the height of the building with the exception of the stories in the vicinity of the outrigger (Fig. 7(g)-(j)). This is because of the stiffening effects of outriggers and increased contribution of higher modes. In contrast, in *Configuration B*, where VCDs are also added in the core wall, inter-story drift ratios are further decreased to a value of less than 0.5%, and coupling beams sustain no plastic rotations (Fig. 7(a)-(d)). This reflects up to 46% reduction in inter-story drift ratios and up to 100% reduction in coupling beam plastic rotations compared with the conventional building (Table 2). This implies that the building remains essentially elastic in Configuration B. Furthermore, significant reductions in floor accelerations, core wall shear forces (with the exception of some minor increase at upper stories in *Y*-dir.), and bending moments are also observed in this configuration (Fig. 7(e)-(j)). In summary, by adding VCDs in Configuration B the performance of the conventional building was shifted from a damage control limit state to one level higher leading to an essentially elastic structure under DE.

The trends in the story-wise distribution of the response indicators under *MCE-level motions* were similar to those observed under DE-level motions. For brevity, only inter-story drift ratios which are mainly used to assess the degree of damage, and core wall bending moment ratios which are used to design foundation of the building are plotted in Fig. 8. Similar to the case of DE-level motions, significant reductions in inter-story drift ratios, compared with the conventional building are observed in both Configurations A and B (Fig. 8(a)-(b)). In particular, Configuration B leads to 39% reductions in maximum drift in *Y*-dir. While Configuration A does not result in significant reductions in core wall bending moments, particularly in *Y*-dir., Configuration B results in significant reductions in bending moments throughout the height of the building (Fig. 8(c)-(d)). In particular, the base bending moment in the *X*-direction is reduced by 20% compared with the conventional building. In summary, under the MCE-level ground motions, Configuration B leads to significant reductions in the levels of damage to the building and provides opportunity for reducing foundation costs because of the significant reductions in base bending moments.

3.3. Effects on Downtime and Financial Losses

PACT (2012)) was used for the loss estimation of the conventional building and for the building with VCDs in Configuration B under DE and MCE-level ground motions. To assess structural losses, slab-to-column and equivalent slab-to-wall connections, and coupling beams were included in the PACT model. For the non-structural element loss assessment, partitions, curtain walls, stairs, suspended ceilings, cold water piping, HVAC system, and fire sprinklers were included. The Normative Quantity Estimation tool in PACT was used to obtain typical quantities of non-structural elements based on the gross floor area. Individual tenant's contents were not included in the model. Peak values of inter-story drift ratios, coupling beam plastic rotations, and floor accelerations from all 7 ground motions were input in PACT and 1,000 Monte-Carlo simulations were performed. Finally, direct repair costs and downtime were evaluated using the median from all 1,000 simulations. Note that downtime was taken as *parallel days* assuming that the repair can be carried out at all floors simultaneously which likely represents a lower bound estimate of downtime. Downtime obtained from PACT, as increased to take into account time for inspection of damage and mobilization of resources for repair, was 124 days under DE and 193 days under MCE for the conventional building and 67 days under DE and 120 days under MCE for Configuration B. Total downtime was then converted into financial losses resulting from business interruption assuming that the owner is renting the property at approx. $3.2 per sq ft. per month, which is typical for a downtown Seattle location. Costs associated with relocation of tenants were not considered in this study. It is observed from the loss estimation results shown in Fig. 9 that due to incorporation of VCDs, total financial losses are reduced by 45% under DE and by 41% under MCE when compared with the conventional building. Furthermore, for DE, negligible cost is associated with the repair of structural components as the structure remains essentially elastic. On the other hand, for MCE, repair cost associated with structural components is reduced by 76%. It is noted that while improved structural performance was the main focus of this study, and the VCD configurations were not optimized with respect to cost, it is expected that the cost of the VCDs would be approximately 1% of the overall structure cost for Configuration A and 2.5% of the overall construction cost for Configuration B. In summary, VCDs lead to significant reductions in losses under both of these seismic hazard levels.

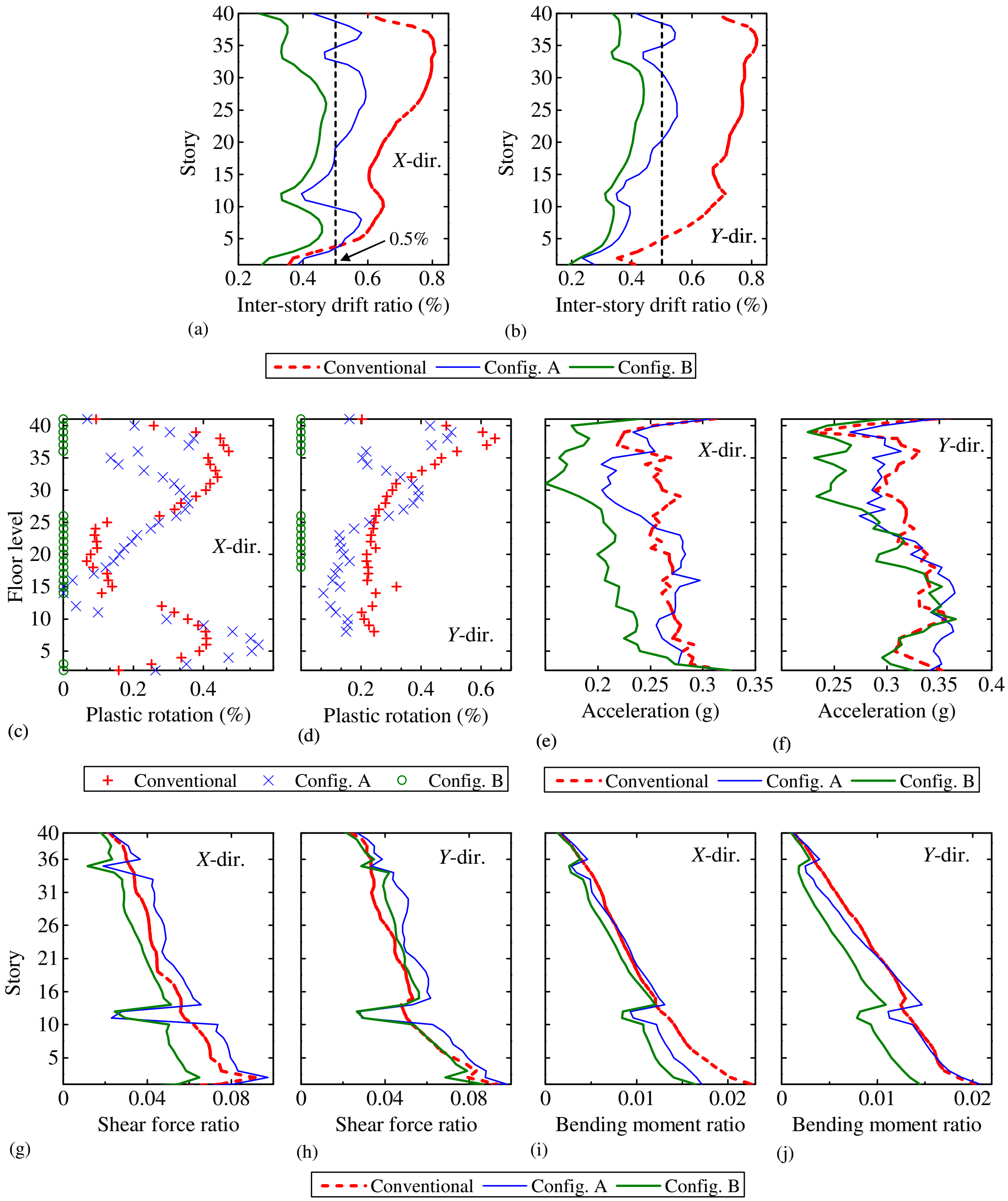

Fig. 7 – Median values of peak response indicators under *DE-level ground motions*: (a)-(b): inter-story drift ratio; (c)-(d): coupling beam plastic rotation; (e)-(f): absolute floor acceleration; (g)-(h): core wall shear force ratio; and (i)-(j) core wall bending moment ratio.

Table 2 – Change in maximum values of peak response indicators in comparison with the conventional building, evaluated from the median response using DE-level motions.

	Inter-story drift ratio		Coupling beam plastic rot.		Floor acceleration		Core wall shear force		Core wall bending moment	
	***X*-dir.**	***Y*-dir.**	***X*-dir.**	***Y*-dir.**	***X*-dir.**	***Y*-dir.**	***X*-dir.**	***Y*-dir.**	***X*-dir.**	***Y*-dir.**
Config. A	-26%	-33%	+18%	-23%	+7%	+5%	-15%	+3%	-17%	+2%
Config. B	-42%	-46%	-87%	-100%	-29%	-7%	-20%	-29%	-34%	-27%

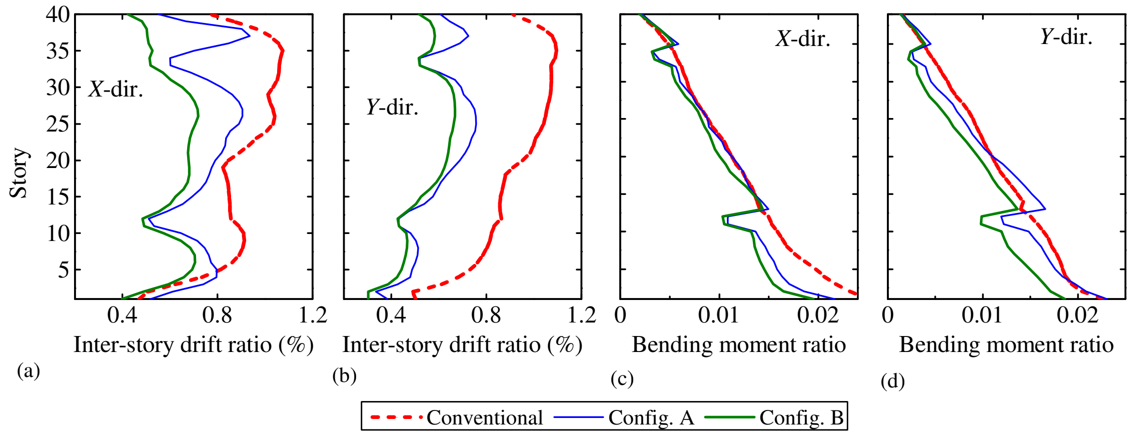

Fig. 8 – Median values of peak response indicators under *MCE-level ground motions*: (a)-(b): inter-story drift ratio and (c)-(d): core wall bending moment ratio.

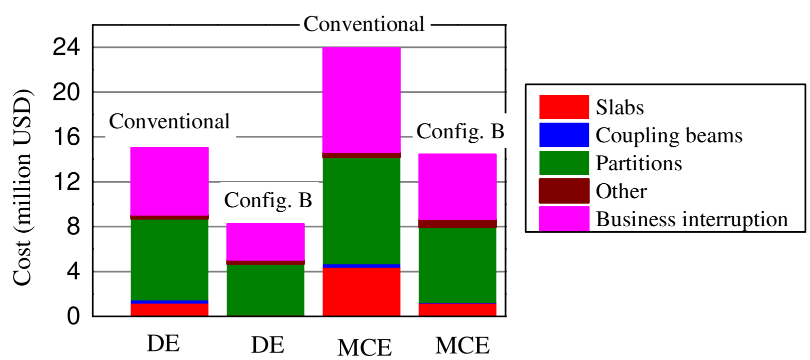

Fig. 9 – Median losses considering direct repair costs and business interruption.

4. Conclusions

Viscoelastic coupling dampers (VCDs) were used to enhance seismic resilience of a 40-story RC building in Seattle, WA, designed using a non-prescriptive performance-based design approach. Two configurations of VCDs were investigated: the first with VCD outriggers at two locations along the height of the building, and the second with core wall VCDs along with steel coupling beams in addition to the VCD outriggers at two locations. Three-dimensional finite element models were developed and response history analyses were carried out under a suite of site-specific ground motions. It was found that the first configuration is effective in reducing inter-story drift ratios but not as effective in reducing coupling beam rotations, floor accelerations, and core wall shear forces, and bending moments, compared with the conventional building. For the second configuration significant reductions in all response indicators were obtained. In particular, median values of peak inter-story drift ratios were reduced by up to 46% under DE and by up to 39% under MCE. More importantly, the conventional building, which suffers moderate damage under DE, remains essentially elastic for this configuration. Loss estimation analysis taking into account direct repair cost of structural and non-structural components and business interruption due to downtime revealed that the second configuration results in 45% reduction in losses under DE and 41%

reduction under MCE compared with the conventional building. It was found that the VCDs greatly enhance seismic resilience of the conventional building by significantly reducing structural as well as non-structural damage throughout the height of the building and by allowing quicker inspection and repair following DE- and MCE-level events.

5. Acknowledgements

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