16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 606
Registration Code: S-X1464628296

SEISMIC RETROFIT OF EXISTING HIGH-RISE BUILDINGS WITH SUPPLEMENTAL ENERGY DISSIPATION DEVICES

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Abstract

The Pacific Earthquake Engineering Research (PEER) Center has expanded its Tall Building Initiative project to include the seismic performance of existing tall buildings. A candidate 35-story steel moment resisting frame with representative details from that era was analyzed. It was identified that the selected building failed to meet the performance objectives suggested by the U.S. code, and had a number of seismic vulnerabilities that endangered its structural integrity under a major earthquake. Therefore, retrofit strategies should be explored to upgrade the building's seismic performance and address its major vulnerabilities. The retrofit intent is to reduce the number of beam-to-column connection failure to an extent that it would not jeopardize the building's overall stability under a basic safety earthquake, level 2 (BSE-2E) per ASCE 41. In addition, the cost-effectiveness of each proposed retrofit strategy will be examined. In this paper, a two-stage retrofit plan is proposed, with a focus on the 2nd stage. In "Stage-1" plan, the widespread brittle column splices are fixed and the heavy exterior cladding is removed. Additionally, "Stage-2" retrofit further improves the structural behaviors by including various supplemental energy dissipation devices in the building. Three devices are selected in this paper, including fluid viscous dampers, viscous wall dampers and buckling restrained braces.

The retrofit design starts by selecting locations to install supplemental devices within architectural and functional constraints. Four exterior frames are chosen considering their less interaction with occupants and interior components. The total effective damping ratio needed at each horizontal direction is estimated based on a target roof displacement at corresponding direction. A damping scale factor is used to facilitate the estimation of total effective damping ratio. To compare the effectiveness of each scheme to meet the target performance objective, an equal quantity of devices are used, and they are installed at the same specified locations in the building. Besides, devices at each location are sized in a way to have a commensurable energy dissipation capacity. Their control effects are compared under BSE-2E using nonlinear response history analysis procedure. Three nonlinear response analyses are used to streamline the design, and the maximum responses are compared. To gain more insights into the cost-effective of each scheme, a probabilistic cost analysis is conducted in the framework of performance based earthquake engineering (PBEE) methodology. Performance assessment calculation tool (PACT) is utilized to estimate the probability of the building having irreparable residual drift and unsafe tagging, and the total repair cost of each retrofit scheme.

Among the three designs using different devices, the fluid viscous dampers are the most effective to control structural responses and reduce the repair cost and other negative impacts after a major seismic event. Consequently, they are demonstrated as the most promising solution to improve the seismic behavior of the selected building.

Keywords: existing tall buildings; retrofit; supplemental energy dissipation devices; PBEE; cost analysis.

1. Introduction

In traditional design where seismic energy is mainly dissipated by irrecoverable inelastic deformation of structural elements, the building safety is maintained at the compromise of components' damage, leading to direct and indirect economic losses. This has been highlighted in recent earthquakes in Chile, Japan, China and New Zealand. As such, the development of seismic protection systems has been spurred, which includes base isolation, active control and passive energy dissipation systems by large [1]. Of these, passive energy dissipation system doesn't require external power source, and is relatively easy to install, and thus considered as a better choice to retrofit existing structures. Three kinds of devices are investigated in this paper: fluid viscous dampers (FVDs), viscous wall dampers (VWDs) and buckling restrained braces (BRBs). Their cost-effectiveness will be discussed and compared in terms of upgrading an existing 35-story Pre-Northridge steel moment resisting frame. Fig. 1 illustrate applications of these devices.

Fig. 1 – Supplemental energy dissipation devices: FVDs, VWDs, BRBs from left to right

2. Case Study Building

The 35-story case study building is an office building, and has complete welded steel moment resisting frames in the longitudinal and transverse directions. The details of the "as-built" building information and numerical modelling are presented in a Lai et al. [2].

A systematic structural evaluation of the case study building indicated that the building failed to meet the performance objectives suggested by ASCE 41-13 [3], and had a number of seismic vulnerabilities. Consequently, proper upgrade methods are desired to enhance its seismic behavior. The intent of the retrofit is to reduce the overall drifts of the structure to a level where brittle fracture of the beam-to-column connections would not seriously jeopardize the overall stability of the structure under the BSE-2E excitations. To achieve this, a "two-stage" retrofit plan is proposed. In "Stage-1", the prevalent brittle column splices are fixed everywhere in the building, and heavy concrete claddings are removed. Nevertheless, these strategies were not sufficient to achieve the target performance goal, and additional strategies are explored. In "Stage-2", various supplemental energy dissipation devices are used in conjunction with strategies used in "Stage-1" for further improvement of structural responses. More information on the original building evaluation could be found in Lai *et al.* [2]. This paper focuses on the "Stage-2" retrofit design, and compares the cost-effectiveness of various supplemental energy dissipation devices in terms of retrofitting existing tall steel buildings.

3. Analysis Method

Nonlinear time history method is used to examine the seismic responses of the existing building using Open System for Earthquake Engineering Simulation (OpenSEES) [4]. The retrofit feasibility study is based on a hazard level corresponding to the BSE-2E, with a recurrence interval of 975 years. Three ground motions are selected at this hazard level to streamline the preliminary design, which are selected based on the closeness of their pseudo-acceleration spectra to the target spectra at the fundamental period of the undamped structure, as shown in Fig. 2. "Stage-2" retrofit starts with a numerical model with "Stage-1" retrofit, i.e., the column splices are fixed everywhere, and concrete cladding are removed. This baseline model is denoted as "as-built" hereafter.

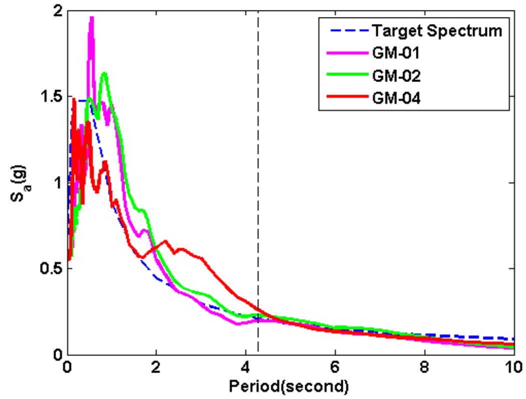
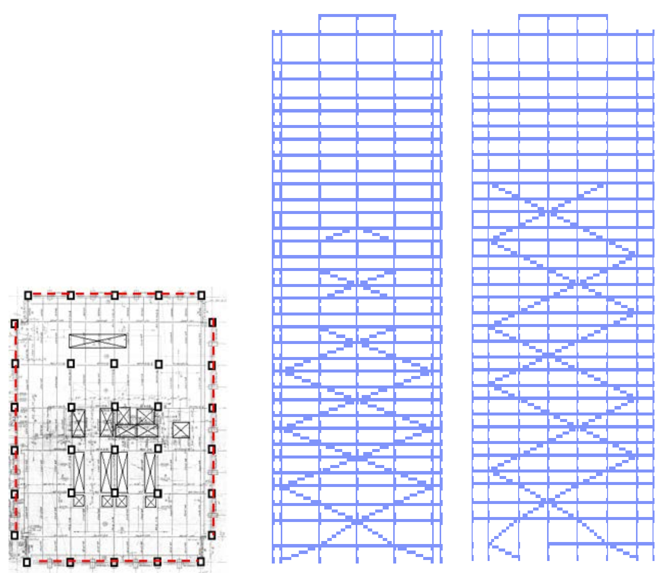


Fig. 2 – Target response spectrum and selected ground motions

4. Retrofit Using FVDs

4.1 Damper locations

To design supplemental energy dissipation devices in an existing building, the first design consideration is selecting proper installation locations. Fig. 3(a) is a sketch of the plane view for a typical floor, where the black boxes indicate column locations. The interior frames are usually adjacent to stairs and elevator locations, and putting dampers there would interfere with office space and egress. Therefore, the perimeter frames were initially selected as the ideal place to add dampers. At first trial, dampers were installed through all stories with three different methods to size dampers (damping constant *C*): (I) uniform, (II) proportional to story shear, and (III) proportional to story stiffness. It was found that sizing dampers proportional to story stiffness was the most efficient and thus selected. Damping exponent α is set to be 0.35 to ensure adequate control effectiveness without excessive damper forces. The initial design is refined by removing dampers in locations with less control effectiveness. Consequently, the refined design concentrated dampers in the lower two-thirds of the building; see Fig. 3(b). Moreover, dampers were distributed across the structure to minimize accumulation of forces transferred to the adjacent columns.



(a) Plane view (b) Elevation

Fig. 3 – Building configuration

4.2 Effective damping ratio

Estimating the overall effective damping (including the intrinsic damping and supplemental damping) needed to reduce the overall drifts and drift concentrations to an acceptable level is critical. A non-iterative approach was used based on researches of Rezaeian *et al.* [5]. In this approach a Damping Scale Factor (DSF) was developed to adjust the 5% damped spectral ordinates to damping ratio ranges between 0.5% and 30%. These factors were based on ground motion predict equations using the entire NGA-W2 earthquake record set. The target roof displacements were selected based on the static pushover curves when the undamped structure abruptly lost more than 70% force resistance capacity. Meanwhile the displacement demands were estimated from the displacement spectrum at BSE-2E event. With the calculated DSF, the desired damping ratios to reach the target roof drift could be estimated using the regression analysis equation in Rezaeian *et al.* report [5], which were 8% for *X*-direction and 13% for *Y*-direction respectively.

4.3 Mathematical modeling of FVDs

General fractional derivative Maxwell model was described by Makris and Constantinou [6] to capture the behavior of FVDs, whereas simplified mathematical model is applicable since FVDs have relative small temperature and frequency dependency [7-8]:

$$F_d = Cv^{\alpha}\cdot sign(v) \tag{1}$$

where C is the damping constant, α is the damping exponent, and *sign (v)* is the sign function of relative velocity of the piston end with respect to the damper housing. In earthquake engineering, α is generally in the range of 0.3 to 1.0 [9]. To model the damper in OpenSEES, a *viscous damper* material is used to represent the damper sub-assemblage: a dashpot and a spring in series. The dashpot resembles the pure damper behavior, as described by Eq. (1); the elastic spring element is representing the driving braces. Researches [10-11] have found that the brace flexibility would influence the damper behavior significantly and shall be fully accounted for. In this study, the total added brace stiffness per story are equal to twice of the story stiffness, which is proven to be adequate stiff to ensure adequate damper effect.

4.4 Design considerations

For a large building presented herein, fairly large dampers are required to achieve target performance goal, which pose great challenges to install them in existing buildings. Issues such as delivering heavy devices to multiple stories and clearing structural/non-structural components would be difficult in construction. Thus alternatives using two dampers per driver, more damped bays at selected stories, or utilizing toggle-brace mechanisms to magnify the effective force of a damping device [12] should be considered. On the other hand, limited performance objectives could be considered to reduce the damper demands.

Another design consideration is the vulnerable columns. With “Stage-1” retrofit, the column tension capacities are less of a concern, but the adequacy of compression capacity remains as an issue. Inclusion of damper could bring down the drift ratios and reduce the axial forces and bending moments in columns. Nevertheless, an excessive accumulation of damper forces on adjoining columns would be likely especially when the structure enters into inelastic range, and large damper forces are generated. Other factors such as the flexibility of connecting elements (e.g., driving braces, girders, connections and columns) would drive the dampers to act more in-phase with peak displacement and add up to the total forces in columns. These two design considerations are not exclusive to FVDs, and are common in other strategies as well.

5. Retrofit Using VWDs

5.1 Mathematical modeling

The locations to add VWDs are kept consistent with the scheme installing FVDs. At locations where wall dampers are implemented, modifications of beam elements are necessary. Fig. 4 illustrates the modifications for a typical frame with a VWD installed in the middle. An additional node is created in the center of the bay for both upper and lower beams. A two node link element connecting these two nodes are generated using a *Kelvin material* model that is assigned in the direction of in-plane movement, as expressed by the following equation:

$$F_d = Ku + Cv^{\alpha} \tag{2}$$

To represent the large bending stiffness of the steel tanks, a rigid offset is assigned for both left and right beams at the connection node, with a total length being 12 feet. The two design parameters of the viscous damper materials: damping constant C, and damping exponent α, are kept consistent with FVDs at each location. Since VWDs are able to provide both additional damping and additional stiffness, an elastic spring is used to represent the steel tank and stiffen the building. Recent tests on VWDs in the United States [13] showed the stiffening of the structure due to wall dampers is about 5%; thus the stiffness parameter: K is set equal to 1000 *k/in* in the building model. Consequently, the overall fundamental period of the case study building is shifted from 4.33 second to 4.10 second.

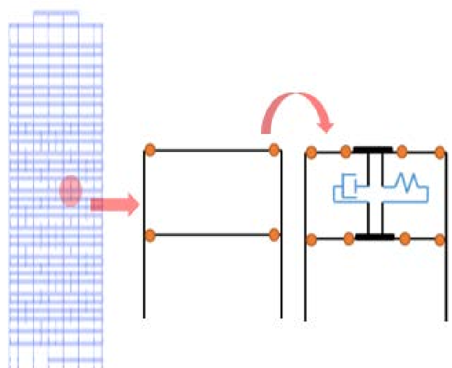
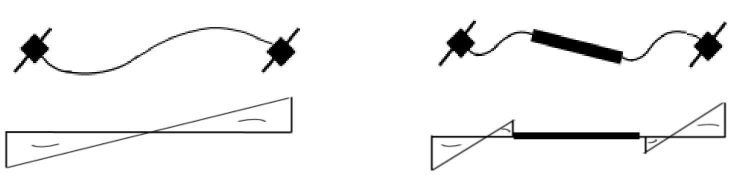


Fig. 4 – VWD modeling

5.2 Design considerations

VWDs could provide more architectural flexibility than the brace-type dampers (e.g., FVDs, BRBs), and its additional stiffness is able to further reduce the displacement demands of the building. Nevertheless, the introduction of VWDs will change the behavior of interaction beam dramatically. The rigid steel tanks will alter the load path of a beam, and change the typically assumed inflection point at mid-span; see Fig. 5. In addition, the moments induced by VWDs are partly in-phase with the moment frame action, which contributed to the increases of the bending and shear demands on the beams. Furthermore, the seismic demands will be increased due to period shifting. In combination of these factors, the shear forces and bending moments at beam ends will be increased, which might contribute to an unexpected beams failure at small deformations. Worse still, a sudden change of deflected shape of the beam will occur at the instance of beam failure, and that might lead to a spike on the deformations and forces of wall dampers. If that is the case, the VWDs may not be able to work properly to dissipate energy after the beams fail.



(a) Deformed shape of beam w/o VWD   (b) Deformed shape of beam w/o VWD

Fig. 5 – Beam deformed shape and bending moment diagram

6. Retrofit Using BRBs

6.1 Mathematical modeling

Similar to VWD_S, BRBs are able to provide both additional stiffness and damping. The fundamental period of the building is reduced from 4.33 second to 4.05 second. To capture the behavior of a BRB, a co-rotational truss element with Giuffre-Menegotto-Pinto material (*Steel02*) assigned in the axial direction is used in OpenSEES. A strain hardening ratio of 0.001, a recommended value to control the transition from elastic to plastic branches and to consider isotropic hardening, is used. An equivalent stiffness is estimated assuming that the energy dissipation capacities between a BRB and a FVD would be close under a same deformation. The model is adequate to capture the primary characteristics of BRBs, e.g., the Bauchinger effect and strain hardening effect.

6.2 Design considerations

Compared to FVDs and VWDs, one major advantage of BRBs is their relative low cost. Besides, the design and analysis procedures are more straightforward, especially in the U.S. where they are considered as ordinary braces. A major concern of using these displacement-dependent devices is the increase of the force demands on structural members, which would be a particular concern for existing buildings. The increased demands are results of period shifting effect and their characteristics to act in-phase with displacement.

7. Analytical Results of Using Different Devices

The global structural responses, damper/BRB behaviors, and column responses are presented for the “as-built” case and retrofitted cases with different devices. The results are the maxima of three nonlinear response analyses set, as required by ASCE 41-13. Each direction is evaluated separately considering different damping ratios used, but only *X*-direction responses are presented for the global responses since the responses in the *Y*-direction follow a similar trend. Most cases went through the entire simulations well, that includes the ground motion duration plus 15-second free vibrations; except for one case with VWDs, where the excitation caused the structure to experience a peak drift ratio exceeding 10% and the simulation was arbitrarily terminated. This criterion is selected by considering the high probability of building collapse at this deformation level, and numerical simulation was not able to capture realistic structural responses after that.

7.1 Global responses

The peak displacement and drift ratio envelopes shown in Fig. 6 and 7 indicate that different devices help reduce the structural deformations by about 20% to 40%. With a same effective damping ratio, all cases bring down the overall roof displacement to a target value, which is 38 inches in the *X*-direction, see Fig. 6. Among three schemes investigated herein, FVDs are the most efficient to remove the concentrated drift ratios and result in a more uniform distribution of the peak deformation. Unfortunately, VWDs and BRBs have limited effects to get rid of the large deformations at lower one-third of the building.

The maximum peak floor accelerations are also examined; see Fig. 8. The “as-built” case exhibits large floor accelerations, which is up to 0.9g at roof level. FVDs help reduce the peak floor accelerations by about 30% throughout the stories, and bring down the peak roof acceleration to 0.7g. On the other hand, VWDs have only limited contributions to suppressing the peak floor accelerations throughout the floors, and there is essentially no reduction on the roof level. Worse still, installing BRBs undesirably increase the acceleration demands at most floor levels, and increase the peak roof acceleration to nearly 1.0g. One major reason accounting for the failure of BRBs to suppress the accelerations lies in the fact that BRBs are acting in-phase with the structural displacements, which increase the forces imposed at each floor level. In addition, the period shifting further amplifies the force demands on the building. A similar trend could be observed for the roof acceleration responses during free vibration period (Fig. 9), where only FVDs contributed to a more rapid decay of vibrations.

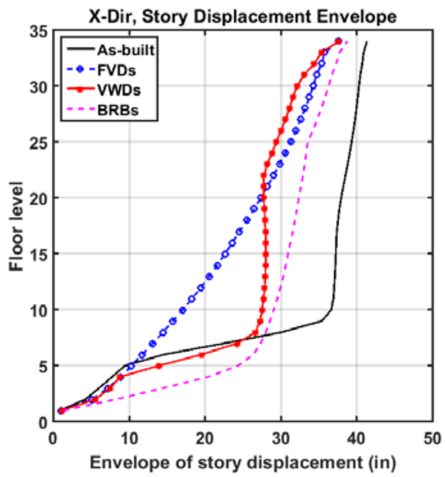


Fig. 6 – Peak story displacement envelope

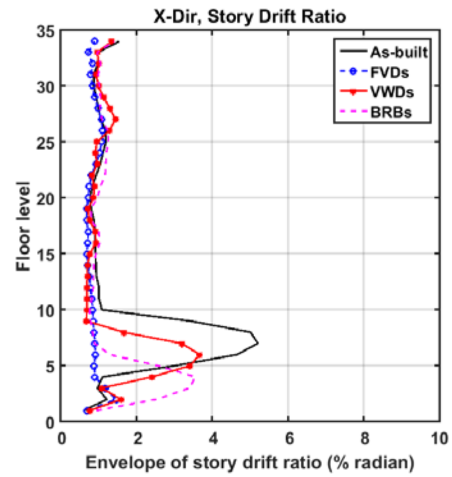


Fig. 7 – Peak drift ratio envelope

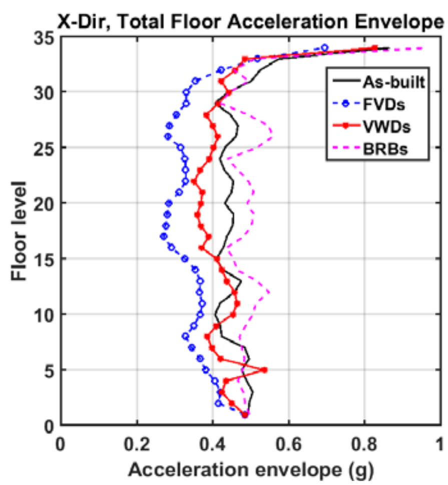


Fig. 8 – Peak floor acceleration envelope

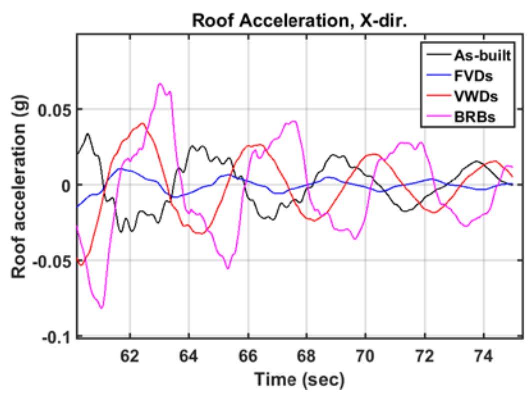
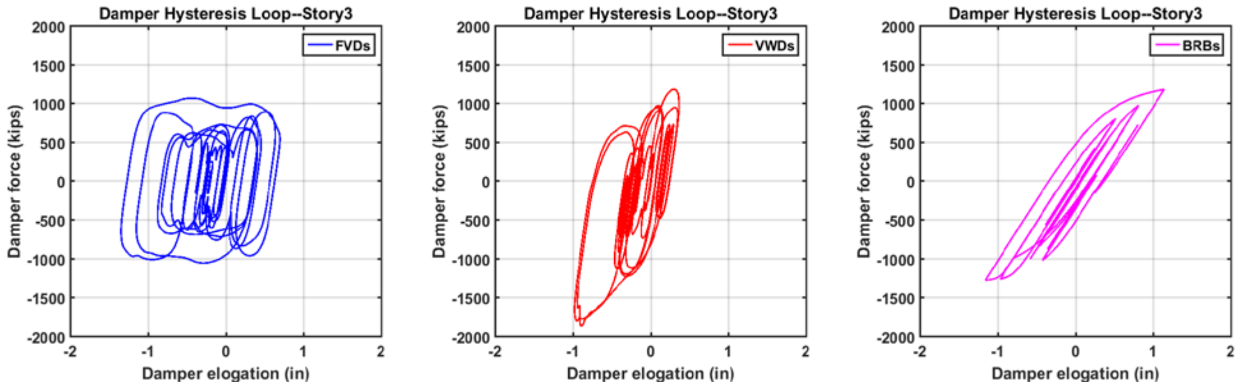


Fig. 9 – Roof acceleration time history

7.2 Damper responses

On the other hand, the damper demands necessary to achieve the control effect should be checked. The hysteresis loops of one damper/BRB at the 3rd story, *Y*-direction are plotted under one ground motion for different schemes; see Fig. 10. Under the same excitation, FVDs mainly exhibit pure viscous properties, as evidenced by the elliptical shape of hysteresis loop. VWDs generally have larger peak forces since the stiff steel tanks at the exterior of wall dampers are able to resist additional lateral forces, though the overall energy dissipation capabilities between VWDs and FVDs are similar. On the other hand, BRBs have a different energy dissipation mechanism than these viscous dampers, and they show a typical bilinear behavior with smaller initial stiffness. The peak deformations are similar for different devices, while the energy dissipation capacities vary, with FVDs having the maximum capacity.

Meanwhile, the damper force demand envelopes are shown for various schemes in Fig. 11. Fairly large force demands are observed for each case, ranging from 1200 kips to 2300 kips. It should be noted that FVDs, that are most effective to suppress the peak deformations and peak floor accelerations, also predicts the smallest peak force demands among the three schemes.



(a) FVDs (b) VWDs (c) BRBs

Fig. 10 – Typical hysteresis loop of different devices

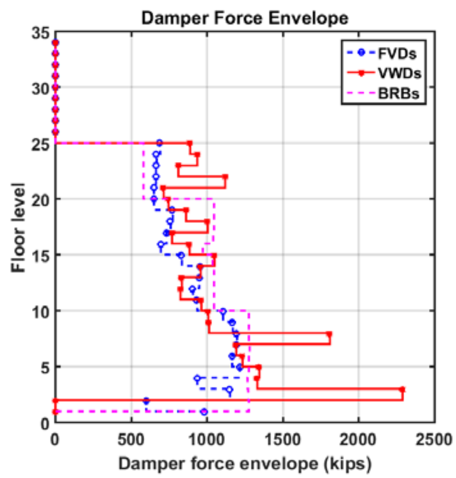


Fig. 11 – Peak damper force envelope

7.3 Column axial demands

The evaluation of the case study building revealed a major vulnerability of the columns. Though the “Stage-1” retrofit eliminated the concerns of brittle column splice rupture/failure, the columns are overloaded in compression and remain as an issue. The problem becomes more critical if we account for the bi-axial bending behaviors and the P-M interaction of the columns. The introduction of supplemental energy dissipation devices is expected to bring down the drift ratios, and thus reducing the axial forces and bending moments demands. However, the additional forces they produce and transfer to adjacent column members might compensate for the reduction. Moreover, the increase of seismic demands due to additional stiffness (i.e., in VWDs and BRBs schemes) would make the issues worse.

To evaluate the influence of supplemental devices on the columns, the axial force demand-to-capacity (*D/C*) ratio envelopes for one group of corner columns (Group 1 in Fig. 12) are investigated. With “Stage-1” retrofit, the brittle column splices are fixed everywhere and thus the columns are estimated to develop the gross section yielding capacities in tension, i.e., $P_t = A_g \times F_y$. On the compression side, the buckling of columns is considered and the lower bound compression capacities are calculated: $P_c = A_g \times F_{cr}$.

The compression *D/C* ratios are indicated with negative signs in Fig. 13. The green line is the compression demands due to gravity force, which constitute about 30% of the total demands. For the “as-built” case, the peak *D/C* exceed 1.0 at floor 6-7, and there are more than half of stories having peak *D/C* ratios larger than 0.5. These ranges are highly likely to experience column failure since the high axial forces would significantly reduce their bending capacities due to the P-M interaction. From ASCE 41 [3], steel columns with an axial compression ratio greater than 0.5 are classified as “force-controlled” group and doesn’t permit any flexural yielding. On the other side, the tension rupture/failure is typically not a concern with all the brittle splices fixed.

For the FVDs case, the peak *D/C* rations are reduced slightly at several floors on tension, while there is no significant reduction on compressive *D/C* ratios. Worse still, neither VWDs nor BRBs are able to address the vulnerable column issues. Actually the axial *D/C* ratios got further increased in these two cases, posing greater danger to widespread column failure. Other strategies to upgrade the column capacities need to be explored.

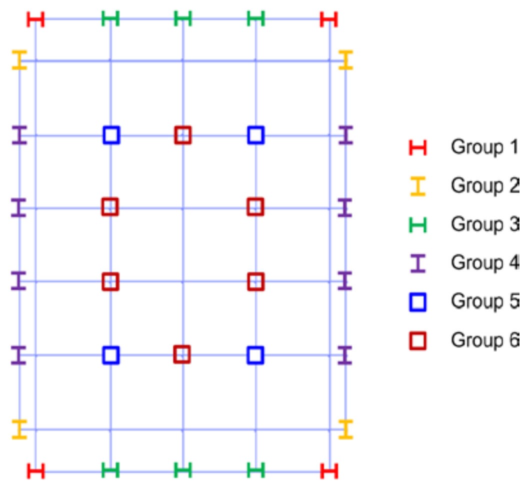


Fig. 12 – Group of columns

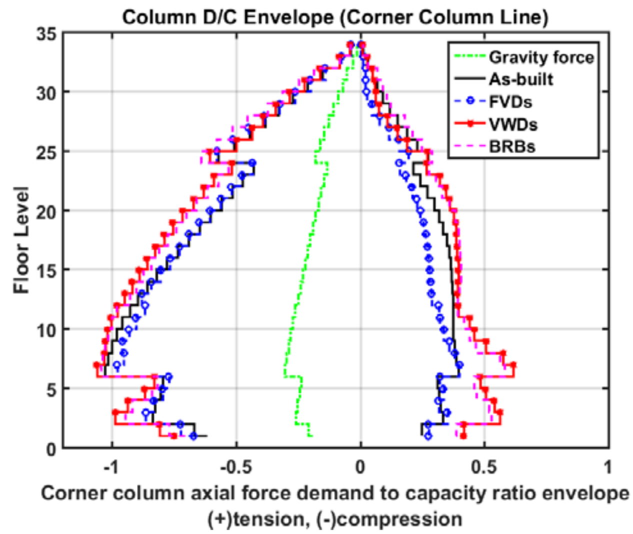


Fig. 13 – Column axial *D/C* ratios

8. Cost Analysis

A cost analysis utilizing the Performance Assessment Calculation Tool (PACT) is conducted in this section. The PACT performs the probabilistic loss calculations from fragility curves of structural and non-structural components and consequence functions for damaged components to estimate repair cost. Four Engineering Demand Parameters (EDPs) are selected to predict the damage states: peak story drift ratios, peak floor accelerations, peak floor velocities, and residual drift ratios. The first three EDPs are extracted from nonlinear response history analyses results, while the residual drift ratios are estimated using an empirical relation with respect to the peak transient drifts following FEMA suggestions [14].

The probability of the building having irreparable residual drifts, the probability of unsafe tagging and their primary contributors are summarized in Table 1. The “as-built” case has large residual drift ratios under BSE-2E, and repair work would be difficult and unsafe. The high chances of the building being irreparable and unsafe indicated that it needs to be torn down and re-built. On the other hand, FVDs successfully bring down the residual drifts, and all these detracting factors are reduced dramatically. A 26.9% of unsafe placard is resulted, with a major contribution from the Pre-Northridge beam-to-column connections and prefabricated steel stairs. Unfortunately, the other two cases: VWDs or BRBs still incur large residual drifts, and are less beneficial to reduce the probabilities of the structure having undesirable responses under BSE-2E.

To assess the particular design, repair loss ratio is used to represent the repair cost, which is defined as the repair cost divided by the building’s replacement cost according to FEMA P-58 [14]. The distribution curve of the loss ratio for different schemes are presented in Fig. 14, with the median values and 90 percentile values identified in Fig. 15. Consistent with previous findings, the “as-built” case has a high chance of having irreparable residual drift, and thus median repair loss ratio is equal to 1.0, indicating a full replacement is needed. The scheme incorporating BRBs brought down the median loss ratio to about 0.084, though its 90 percentile value hits 1.0. Installing VWDs is the least efficient to suppress the peak structural responses, and leads to a median repair loss ratio equaling to 1.0. On the contrary, FVDs significantly enhance the building behaviors, reducing the median and 90 percentile repair loss ratio to 0.047 and 0.071, respectively.

As a further investigation, the initial costs of various supplemental energy dissipation devices are checked. Based on available online resource and information from experienced engineers, an estimates of the initial cost for different schemes are summarized in Table 2. As expected, BRBs are cheapest and VWDs are the most expensive among these three. In construction work, other factors such as fixing existing structural elements (e.g., beam-to-column connections) to create adequate load transfer path, transporting heavy devices to higher floors, and removing non-structural components to accommodate the devices need to be fully accounted for, which might result in a total additional device cost to be 5 to 10 times bigger than their initial cost. Nevertheless, the significantly better structural performances when the building is retrofitted with FVDs could save a great amount of the post-earthquake repair work, and reduce the overall financial losses when an earthquake happens.

Table 1 – Loss estimates

Scenario	Probability of irreparability	Probability of unsafe placard
As-built	94.3%	98.3%
FVDs	0.6%	26.9%
VWDs	66.0%	70.6%
BRBs	45.5%	60.6%

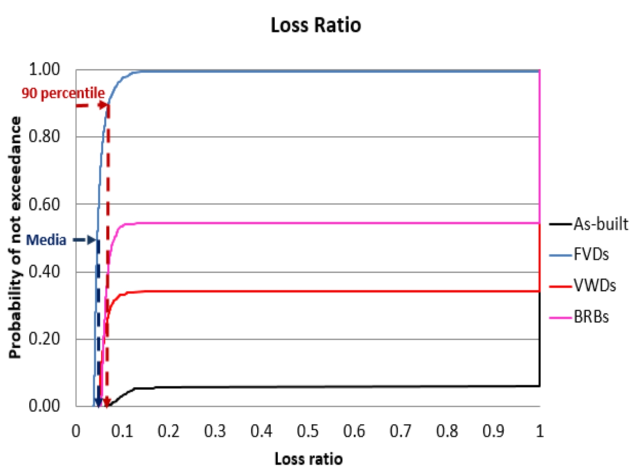


Fig. 14 – Loss ratio curve

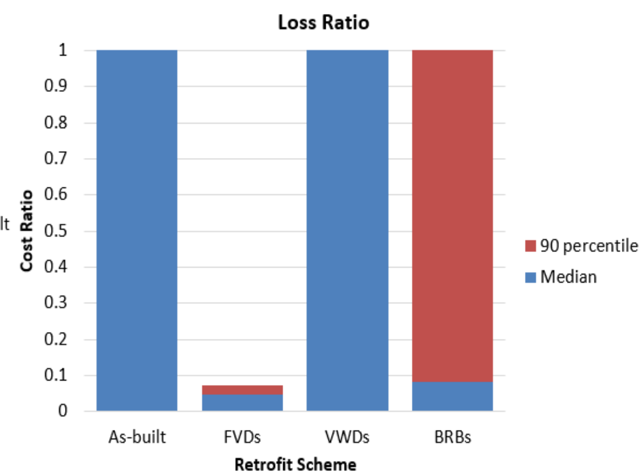


Fig. 15 – Median and 90 percentile loss ratio

Table 2 – Initial cost of retrofit schemes

Scheme	Initial Cost (*$M*)
FVDs	6.4
VWDs	8.4
BRBs	1.7

9. Conclusion

A representative Pre-Northridge high-rise steel moment resisting frame was evaluated using ASCE 41 procedures. Several major structural vulnerabilities were identified, which solicited the cost-effective study of possible retrofit strategies. A two-stage retrofit plan is proposed. In “Stage-1”, the column splices were fixed everywhere, and the exterior heavy claddings were removed. “Stage-2” plan focuses on using supplemental energy dissipation devices to further enhance the structural behaviors, and is examined in detail in this paper.

Three devices are investigated: FVDs, VWDs and BRBs. The design starts by the case with FVDs. Four perimeter frames are selected to install these devices so that the interaction of occupants and interior components could be minimized. The total effective damping ratios are estimated to achieve the target roof displacements at each horizontal direction. A refined damper design is proposed where dampers are installed only in locations with better control effectiveness. These locations are kept consistent for the other two schemes incorporating VWDs or BRBs. In addition, the mechanical properties as three devices are selected based on a same energy dissipation assumption.

The maximum results of three nonlinear response history analyses set were selected for investigations. The global responses showed that the FVDs are the most effective to bring down the drift concentrations at floor level 4 to 10, and resulted in a more uniform distribution of the peak deformations. The resulted peak drift ratio in the case of FVDs is less than 1.5%, which essentially eliminates the beam-to-column connections failure under BSE-2E event. FVDs scheme also turns to be the most efficient to suppress the peak floor accelerations and results in a more rapid decay of the structural vibrations. On the other hand, VWDs cause additional problems on the existing beams, and these wall dampers are not able to function properly, nor provide desired structural control effect. The displacement-dependent BRBs are acting more in-phase with structural displacements, and thus increasing the force demands on the existing building. Besides, the stiffening effect of both VWDs and BRBs induce additional seismic demands, putting extra pressure on the existing structural elements. As such, neither of the latter two schemes could achieve the target retrofit objective. In addition, the behavior of dampers or BRBs are checked. It is shown that fairly large devices are required for all schemes, while the required FVDs turn to be the smallest among them despite of their optimal control effects. Moreover, cost analysis is conducted following FEMA P58. The results are quite consistent with the structural analyses results, showing that FVDs are the more effective to bring down the detracting factors and lead to much smaller economic losses after a major earthquake event.

Several design considerations are presented for each scheme. One general issue prevalent in all cases is the vulnerable columns. Even with the brittle splices fixed, the columns are overloaded in compression and additional upgrading methods should be explored.

In summary, among three energy dissipation devices investigated, FVDs have the least interaction with structural members, and are able to introduce additional damping without significantly increasing the structural demands on the vulnerable columns and beams. Therefore, they are viewed as the most promising solution to improve the structural behavior and reduce the economic losses of the case study building.

10. Acknowledgement

This paper is supported by Pacific Earthquake Engineering Research center (PEER) as part of its Tall Building Initiative and Next-Generation Attenuation Relation programs. The assistance of Frank McKenna, Andreas Schellenberg and Prof. Dimitrois Lignos in developing OpenSees numerical models is gratefully acknowledged. The authors would also like to express gratitudes to Prof. Kazuhiko Kasai, Dr. Kit Miyamoto and Dr. Amir Gilani of Miyamoto International, Dr. Amarnath Kasalanati of Dynamic Isolation Systems, Jim Malley of Degenkolb Engineers and Rob Smith of Arup for sharing their valuable expertise and advice. In addition, the price information of damper devices provided by Shanghai Research Institute of Materials is greatly appreciated.

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