Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 300

SEISMIC MITIGATIION OF BUILDING STRUCTURAL SYSTEMS USING PASSIVE DAMPERS

David P Thambiratnam[1]

ABSTRACT

This paper treats the seismic mitigation of medium rise frame-shear wall structures and building façade systems using passive damping devices. The frame shear wall structures have embedded viscoelastic and friction dampers in different configurations and placed in various locations in the structure. Influence of damper type, configuration and location are investigated. Results for tip deflections which provide an overall evaluation of the seismic response of the structure, are determined. Seismic mitigation of building façade systems in which visco-elastic dampers are fitted at the horizontal connections between the facades and the frame, instead of the traditional rigid connections, are also treated. Finite element techniques are used to model and analyse the two structural systems under different earthquake loadings, scaled to the same peak ground acceleration for meaningful comparison of responses. Results demonstrate the feasibility of these techniques for seismic mitigation.

Introduction

Seismic mitigation of structural systems can be provided by mechanical energy absorbing devices or dampers. There are two types of dampers – active and passive. Since active dampers require a power supply which may be disrupted during a seismic event, passive dampers are preferred in providing seismic mitigation. Different types of manufactured passive dampers available in the market are reviewed by (Constantinou 2000). These dampers have different dynamic characteristics and will affect the seismic response of structures differently. Viscoelastic (VE) dampers which dissipate energy at all levels of deformation and over a broad range of excitation frequencies and friction dampers that dissipate energy only when the slip force is reached and exceeded are used in this paper.

Two structural systems are considered (i) 18 storey frame-shear wall structure with dampers embedded in cut-outs of shear walls and (ii) 12 storey building facade systesm with VE dampers at the horizontal connections, instead of the traditional rigid connection. With such rigid connections facade systems have been extremely vulnerable to seismic events and have failed

[1] Professor, School of Urban Development, Queensland University ofTechnology, Brisbane, Australia

with devastating effects more often than the buildings. Finite elemnt models of the two structural-damper systems are analysed under different earthquake records, all scaled to a common peak ground acceleration (PGA) to enable meaningful comparison of results. The response parameters are tip defelctions and acceleerations in the frame shear wall structure and differential displacements between facade and frame and inter-storey drifts in the building facade system.

Frame Shear Wall Structure

The two-dimensional 18-storey concrete frame shear wall structure shown in Fig. 1 is considered. The shear walls are modelled using shell elements of designation S4R5, having 4 nodes per element and five degrees of freedom at each node. The dimensions of the shear walls are 6m wide and 0.4 m thick while the columns and beams have cross-sectional dimensions of 0.75 x 0.75 m and 0.75 x 0.45 m respectively and the beam spans are 6.0m. The height between storeys is 4.0 m giving an overall height of 72 m. The natural frequency of the undamped structure is 0.614 Hz and in the range 0.570 - 0.650 Hz when fitted with dampers. These values are within the range of dominant frequencies of all the seismic records chosen in this investigation (varying from 0.58 Hz to 1.07 Hz, as will be seen later) and hence this study treats the structural response under a range of seismic excitations including a resonant range. Six different damper placements as shown in Fig. 1 are used to study the influence of location on their seismic response. The undamped structures is also analysed in order to compare results The concrete has a compressive strength = 32 MPa, Young's modulus = 30,000 MPa, which assumes predominantly elastic response with little wall cracking, Poisson's ratio = 0.2, and density = 2500 kg/m^3. Structural steel was used to model friction dampers with Poisson's ratio = 0.3, density = 7700 kg/m^3 and coefficient of friction = 0.25. After preliminary convergence study the concrete shear walls were modeled with 2016 S4R5 shell elements

Dampers

The displacement dependent fiction damper is modeled as a tube sliding inside another tube with friction contact between the sliding surfaces. The extended version of the classical isotropic Coulomb friction model in the computer program ABAQUS is used (Marko 2006). The velocity dependent VE damper is modelled as a linear spring and dash-pot in parallel (known as the Kelvin model) where the stiffness k_d and the damping C_d are represented by a spring and a dashpot respectively. General theory of these dampers are found in (Abbas 1993) while calcualtions for k_d and C_d used in this paper can be found in (Marko 2006). The hybrid friction-VE damper, used in the frame shear wall structure is a combination of the friction and VE dampers in series. Different damper configurations consisting of diagonal friction and VE dampers, chevronbrace (horizontal) friction and VE dampers, hybrid friction -VE dampers and lower toggle VE dampers (detials given below) are considered in the frame shear wall structures

Friction Dampers

A friction damper located within shear wall can be seen in Fig. 2a where a 3.5 m wide by 3.5 m high wall section has been cut out and replaced by a diagonal friction damper. The damper was modeled as a pair of tubes each with a thickness of 50 mm, and with one tube placed inside the other. The outer tube had an inner diameter of 200 mm and length 3.75m and was modeled using 264 S4R5 shell elements. The inner tube had an outer diameter of 198 mm and length 3.75 m and was modeled with 252 S4R5 shell elements. The radial clearance between the tubes was 1mm and the contact area in the unloaded state was 3.71 m^2. The connection between each tube and the shear wall was through a MPC (Multi-Point Constraint) Pin type element. A MPC Slider type connecting element enabled frictional sliding between the tubes in a determined direction. The friction dampers in all other configurations were similar except for their lengths and hence the size of the cut-outs in the shear walls. Further detauils can be found in (Marko 2006).

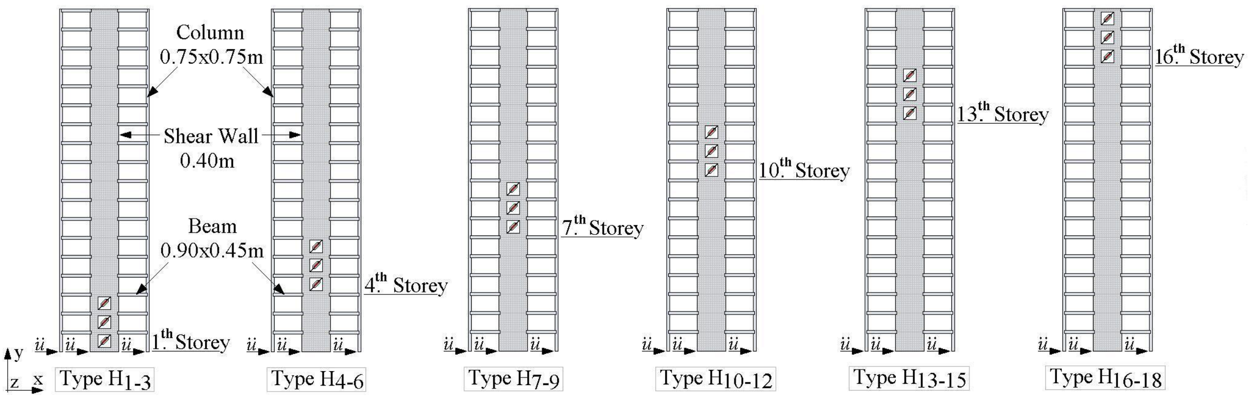

Figure 1. Damper placements in 18-storey frame-shear wall structures

VE Dampers

A diagonal VE damper located within the cut out of the shear wall can be seen in Fig. 2b. The properties of the damper for the 18-storey structural model were calculated to be k_d = 10 x 10^6 N/m and C_d = 50x10^6 Ns/m for the loading frequency f = 0.614 Hz, which corresponded to the fundamental frequency of this structure model. VE dampers in other configuations were similarly modelled (Marko 2006).

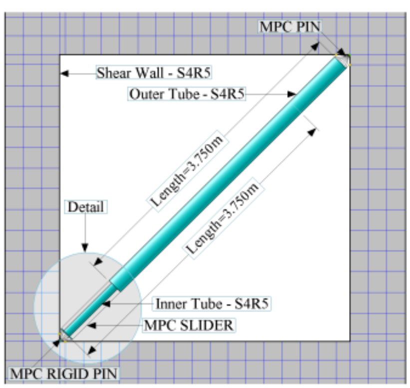

Figure 2a. Friction damper.

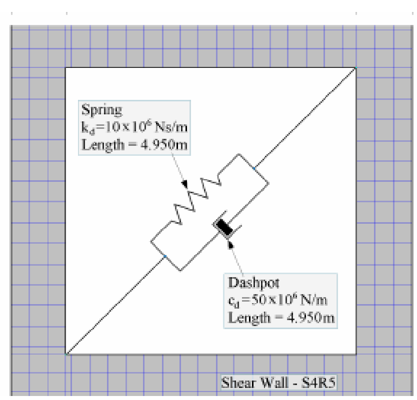

Figure 2b. VE damper.

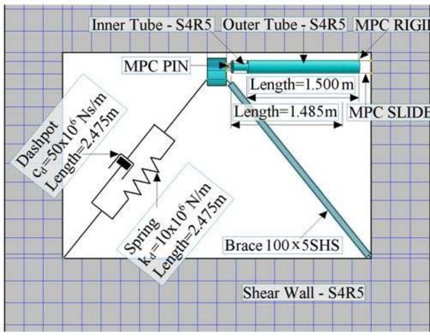

Figure 3a. Hybrid friction-VE damper.

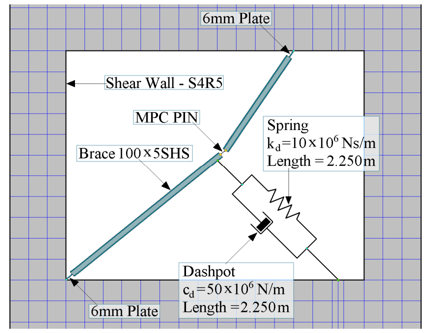

Figure 3b. Lower toggle-VE damper

Hybrid friction-VE dampers

The hybrid friction-VE damper was created to represent 50% of the damping force of the diagonal VE damper, and 66.6% of the damping force of the chevron brace friction damper. The VE part of the hybrid damper had a lenth of 2.475m and was oriented at 45^0 with one end attached to a steel holder placed in the middle of the upper edge of the cut out, and the other end attached to the lower left-hand corner of the cut out, as shown in Fig. 3a. This hybrid damper is expected to utilise the desirable features of both the VE and friction components.

Lower toggle VE dampers

In the recent past several new configurations of passive energy dissipation devices have emerged with innovative mechanisms to amplify displacement and hence lower input force demand in the energy dissipating devices. This paper considers one such system – the lower toggle as shown in Fig. 3b. This lowere toggle VE damper is oriented at 45^0 to the horizontal with its length of 2.262 m with one end attached to the lower arm of the steel holder and the other end attached to the lower right-hand part of the cut out. In this configuration, the arms of the brace assembly were created from 100 x 5 SHS and these arms were connected to each other by 6mm pre-bent plate and the connection to the shear wall was by MPC Pin.

Seismic Records

Five well known seismic records are used in this study, scaled to a peak acceleration of 0.15 g, which is a reasonable average value and suitable for Australia with low seismic activity. These seismic records with different dominant frequencies and strong motion durations were chosen to study the effectiveness of the mitigation technique under a suite of seismic recods. They are applied to the frame shear wall structure for the first 20s. Duration of the strong motion and range of dominant frequencies were evaluated by Welch's method (Welch 1967) based on Fast Fourier Transform Techniques, using the computer program MATLAB Version 6.5. The seismic records with recording stations wherever known within paranthesis are: Imperial Valley (El Centro 1940) with strong motion during 1.5-5.5 secs and dominant frequencies in the range 0.39-6.39 Hz, Hachinohe (1994) with strong motion during 3.5-7.5 secs and dominant frequencies in the range 0.19-2.19 Hz, Kobe (1995) with strong motion during 7.5-12.5 secs and dominant frequencies in the range 0.29-1.12 Hz, Northridge (1994) with strong motion during 3.5-8.0 secs and dominant frequencies in the range 0.14-1.07 Hz and San Fernando (Lankershim

Blvd. 1971) with strong motion during 4.5-9.5 secs and dominant frequencies in the range 0.58-4.39 Hz. For the building facade system, Imperial Valley (El- Centro), Kobe and Northridge seismic recordes were used, scaled to a peak acceleration of 0.2g which is also an average peak value suitable for Australia with low seismic activity. These are applied for the first 20s.

Results and Discussion

The objective of the present study was to provide seismic mitigation for the frame shear wall structure and hence only the tip deflections were evaluated. Evaluation of tip deflection is a reasonable measure of the overall effect of the earthquake and hence any reduction in tip deflection represents a worthwhile reduction in overall seismic design force. Results show that this reduction is dependent on the complex characteristics of the time histories used for assessment and hence the benefits can only be legitimately assessed if the analysis is carried out for the suite of time histories, as done herein. Fig. 4 illustrates the deflection time histories of the undamped structure and the structure fitted with diagonal VE dampers in the lowest three storeys, under the El Centro seismic record. From these graphs as well as from numerous other results it was evident that embedded dampers are able to effectively provide seismic mitigation.

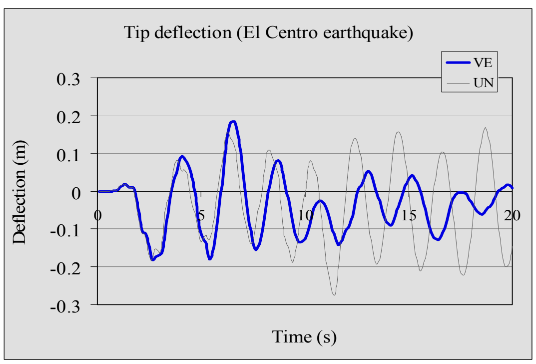

Figure 4. Tip deflection time histories

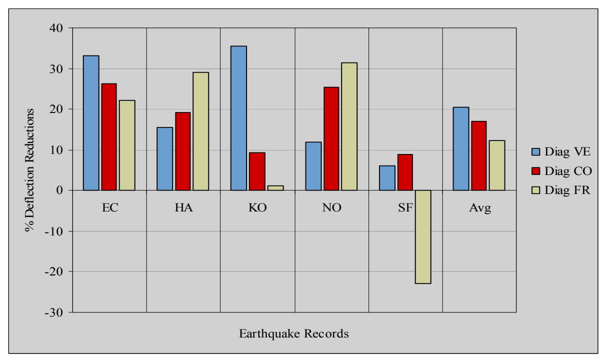

Figure 5. Comparison of deflection mitigation

The undamped structural model was developed and analysed in order to compare its results with those from the damped structures. Results for the maximum tip deflections of the undamped structure experienced under the five earthquake excitations are presented in Table 1. The percentage reductions in the peak values of the tip deflections experienced by the structures fitted with all the damping systems are presented in Table 2.

Table 1. Maximum tip deflections of the undamped structure.

Earthquake	El Centro	Hachinohe	Kobe	Northridge	S. Fernando
Deflection (m)	0.275	0.464	0.163	0.245	0.130

The diagonal friction dampers display a wide range of efficiency, with significant reductions in most cases. The greatest average reduction of 23.6% occurred under the Hachinohe earthquake. Some increases were also experienced, especially under the San Fernando earthquake. This may be attributed to inadequate compensation for removed stiffness and/or partial resonance of the damped structure. In terms of damper placement, the highest average tip deflection reduction was achieved by the structure with dampers fitted in the top storeys, while

Table 2. % Reductions in tip deflections for all damping systems

Dampers	Model	El Centro	Hachinohe	Kobe	Northridge	S. Fernando	Average
Diagonal Friction Dampers	H 1-3	14.55	13.48	13.50	-11.02	8.40	7.78
	H 4-6	13.09	19.15	-7.36	11.43	1.53	7.57
	H 7-9	17.09	28.37	-6.75	16.73	-37.40	3.61
	H 10-12	22.55	26.24	-6.13	6.94	-41.22	1.67
	H 13-15	24.36	25.53	-4.29	16.33	-34.35	5.52
	H 16-18	22.18	29.08	1.23	31.43	-22.90	12.20
	Average	18.97	23.64	-1.64	11.97	-20.99	6.39
Diagonal VE Dampers	H 1-3	33.09	15.60	35.58	11.84	6.11	20.44
	H 4-6	33.82	12.06	28.22	13.47	6.87	18.89
	H 7-9	20.36	8.51	24.54	8.16	5.34	13.38
	H 10-12	12.73	4.96	14.72	5.31	3.82	8.31
	H 13-15	8.00	2.13	3.07	4.90	3.82	4.38
	H 16-18	7.64	2.84	-10.43	3.67	-1.53	0.44
	Average	19.27	7.68	15.95	7.89	4.07	10.97
Chevron Friction Dampers	H 1-3	4.71	-3.15	17.35	7.98	14.79	8.34
	H 4-6	6.67	2.36	13.27	9.24	13.38	8.98
	H 7-9	7.45	5.51	12.24	8.40	10.56	8.83
	H 10-12	10.98	9.45	6.63	9.66	5.63	8.47
	H 13-15	15.29	14.17	1.02	12.61	4.23	9.46
	H 16-18	12.55	13.39	7.14	10.50	-0.70	8.58
	Average	9.61	6.96	9.61	9.73	7.98	8.78
Chevron VE Dampers	H 1-3	5.49	-0.79	17.35	6.30	12.68	8.21
	H 4-6	6.67	6.30	14.80	7.14	9.86	8.95
	H 7-9	7.06	9.45	9.18	8.40	5.63	7.95
	H 10-12	10.20	12.60	8.16	10.08	2.11	8.63
	H 13-15	10.59	14.17	4.08	10.50	0.00	7.87
	H 16-18	12.55	13.39	8.67	9.24	-0.70	8.63
	Average	8.76	9.19	10.37	8.61	4.93	8.37
Hybrid Dampers	H 1-3	1.81	-4.68	19.69	6.45	16.70	7.99
	H 4-6	7.47	-0.62	10.09	9.81	14.66	8.28
	H 7-9	4.64	4.25	12.62	7.29	10.56	7.87
	H 10-12	8.27	9.11	5.04	8.55	7.14	7.62
	H 13-15	10.70	10.74	2.52	9.39	5.78	7.82
	H 16-18	8.68	9.93	9.59	8.97	3.73	8.18
	Average	6.93	4.79	9.92	8.41	9.76	7.96
Lower Toggle Dampers	H 1-3	8.94	-0.18	30.22	9.99	23.74	14.54
	H 4-6	4.58	1.41	19.33	17.39	23.07	13.16
	H 7-9	8.15	3.80	21.81	13.28	21.06	13.62
	H 10-12	10.52	10.16	20.32	13.69	17.05	14.35
	H 13-15	13.29	14.93	11.91	17.80	11.03	13.79
	H 16-18	15.67	17.31	8.94	17.39	-1.68	11.52
	Average	10.19	7.90	18.75	14.92	15.71	13.50

the lowest average reduction occurred for the structures with dampers placed in the storeys 10 to 12. The results obtained under the El Centro, Hachinohe and Northridge earthquakes fully support Hanson's theory (Hanson et al. 1993), which recommends placement of dampers at levels of maximum interstorey drift. On the other hand, with the Kobe and San Fernando earthquakes, a high efficiency occurred only with dampers fitted in the lowest storeys. The overall performance of the diagonal VE dampers was significantly high, but the range of results remained wide. The average tip deflection reductions varied from 4.1% under the San Fernando earthquake, to 19.3% under the El Centro earthquake. The best performance occurred when the dampers were placed in the lowest storeys, while moving them towards the top of the structure resulted in a gradual decrease in tip deflection under all earthquake excitations.

The performances of the chevron brace friction dampers under all earthquakes were within a very narrow range (7.0-9.7%). It was evident that under Kobe and San Fernando earthquakes the highest efficiency occurred when these dampers were placed in the lowest storeys and decreased rapidly as the dampers were moved towards the top of the structures. But under El Centro, Hachinohe and Northridge earthquake excitations, damper efficiency is increased when they are moved to regions with large inter-storey drifts. The performance of chevron brace VE dampers under the El Centro, Hachinohe, Kobe and Northridge earthquakes was consistent, whereas its performance under the San Fernando earthquake was adequate only in the lower storeys. These dampers followed trends sismilar to chvron brace friction dampers with regards to damper placement. The hybrid friction-VE dampers achieved satisfactory average reductions under the Kobe, Northridge and San Fernando earthquakes, whereas the reductions under the El Centro and Hachinohe earthquakes were less. In the case of the El Centro, Hachinohe and Northridge earthquakes, the highest average deflection reductions were obtained when the dampers were placed in the storeys 13 to 15 while in the case of the Kobe and San Fernando earthquakes, it was when the dampers were placed in the lowest storeys. In the case of the lower toggle VE dampers the highest average reduction of 18.8% was obtained under the Kobe earthquake, whereas the lowest average reduction of 7.9% occurred under the Hachinohe earthquake. In the cases of the El Centro and Hachinohe earthquakes, the highest tip deflection reductions occurred when the dampers were placed in the uppermost storeys, while a gradual decrease was experienced as the dampers were moved towards the bottom of the structure. A reverse trend occurred under the Kobe and San Fernando earthquakes. In the case of the Northridge earthquake, the performance remained relatively consistent for the all placements.

Table 3. % Reductions in tip deflections and accelerations with combined damping system

	El Centro	**Hachinohe**	**Kobe**	**Northridge**	**S. Fernando**	**Average**
%Defl. Red.	26.16	19.18	7.36	25.31	6.92	16.99
%Accel. Red.	15.55	25.85	14.80	13.28	8.82	15.66

Based on the the above findings, the effect of a combined damping system consisting of a diagonal friction damper in the 16th storey (which is the average storey number for maximum interstorey drifts) and a diagonal VE damper in the 1st storey (diag CO) was invetigated. The results for percentage reductions in tip deflections and tip accelerations are presented in Table 3. Fig. 5 compares the average tip defelction reductions of this system with those of the diagonal friction and VE dampers under all five seismic records. It is evident that the combined damping system can achieve significant reductions in both response parameters under all earthquake excitations, eventhough it consisted of only two dampers. The above results were obtained for the chosen 18- storey shear wall frame structure, but they may be useful in the seismic mitigation of other similar structures.

Building Facade System

A 12-storey, 4 bay structural model with a storey height of 4 m and bay span of 8m, as shown in Fig. 6 is chosen for the investigation. The columns and beams of the frame have cross-sectional dimensions of 0.6 x 0.6 m and 0.65 x 0.6 m, respectively to support the gravity loads. The fundamental natural frequency of this structure was 0.84 Hz and this value is within the strong motion frequency range of the selected seismic records. Uniformly distributed loads of 40 kN/m were applied to all the beams except the roof beam which had a load of 34 kN/m.

Pre-cast concrete is chosen for the facades as they are common in Australia and world wide The facade panels were placed in the second storey and onwards up to the 12 storey at 0.05 m distance from the building frame. In this study the dimensions of the facade panels were kept as 7.9 m wide, 3.9 m high so as to accommodate the connections in the computer model. Concrete used for the frame and the precast concrete facades had the following properties: compressive strength = 32 MPa, Young's Modulus = 30,000 MPa, density = 2400kg/m3 and Poisson's ratio = 0.2.

Connections

Each facade panel had 4 vertical connections at the beam ends and 4 horizontal connections at the column ends. VE dampers were chosen as the horizonatl connectors as they are efficient in seismic mitigatiin and easy to model. They are modelled by spring (representing stiffness) and dashpot (representing damping) in parallel at the column façade connections. Properties of the VE damper connections were calculated as stiffness k_d = 20 x 10^6 N/m and damping coefficient C_d = 35 x 10^6 Ns/m for the frequency 0.84 Hz, which corresponded to the fundamental frequency of the 12 storey structure model. The vertical connections consisted of springs with a stiffness of $30x10^6$ N/m, which could support the weight of the façade panel and which corresponded to the stiffness of typical facade connections. The horizontal connections in the un-damped structure were also springs with a stiffness of $20x10^6$ N/m. Further details on connection properties can be found in (Hareer 2007).

Analysis and results

One dimensional frame elements were used to model the beams and columns and two dimensional plane stress elements were used for the facade panels. The building facade system was analysed in turn under the action of the El Centro, Kobe and Northridge seismic records (descibed earlier) which were applied in the x-direction at the base of the structure as shown in Fig 6. The objective of this part of the study was the seismic mitigation of the building facade system, with a focus on preventing facade faulure. With this in mimd, the two important response parameters obtained from the results of the analysis are: (i) differential displacements between facade and frame and (ii) inter-storey drifts of the frame.

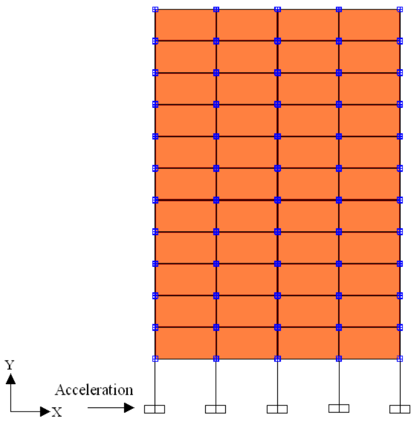

Figure 6. Building facade system

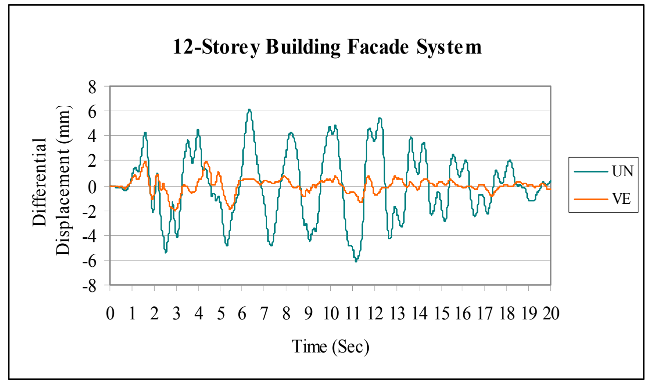

Figure 7. Differential displacament response

Fig. 7 shows the time history responses of the differential displacments of the damped and undamped structure, under the El Centro earthquake record, while Fig. 8 shows the maximum values of these differential displacements under all theree earthquake records. With the introduction of VE damping connections to the structure, the differential displacements are reduced by an average of 80 %. Fig. 9 shows the maximum values of the interstorey drifts and it is evident that these trends are similar to those of the differential displacements. With the insertion of the VE damping connections, the interstorey drifts are reduced by an average of 78% under the 3 seismic records.

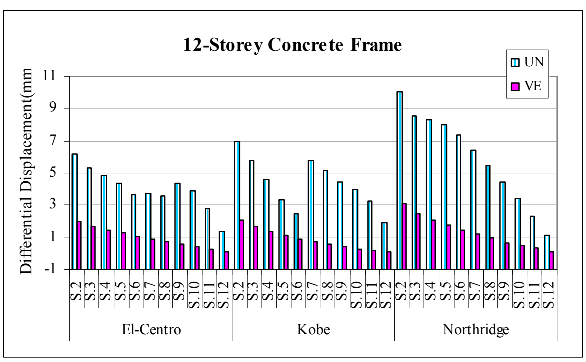

Figure 8. Maximum differential displacements

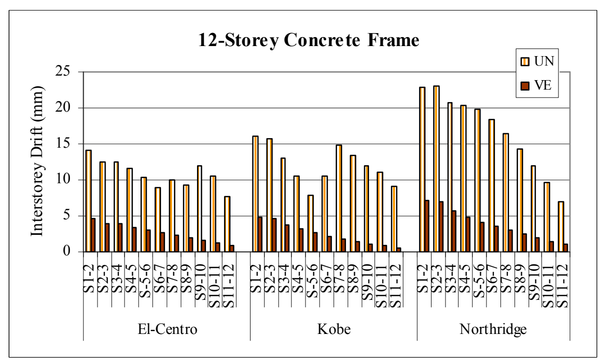

Figure 9. Maximum inter-storey drifts

Conclusion

This paper treated the seismic mitigation of an 18 storey frame shear wall structure with embedded dampers and a 12 storey building facade systems with damapers at the horizontal connections, under different seismic excitations. The frame shear wall structure had friction and VE dampers in different configuarations and at different locations in cut outs of the shear walls. Results for the frame shear wall structure show that friction dampers, in the large majority of cases, surpassed the VE dampers in their ability to reduce the intensity of the initial strong strikes. The VE dampers on the otherhand, gradually decreased the deflection of the structure. The performance of the friction dampers increased with higher interstorey drift, while the best performance of VE dampers was achieved when placed in the lowest storeys. The diagonal friction dampers performed better under the earthquakes which produced higher deflections of the structure. The chevron brace dampers which had only 66.6% of the damping force of the diagonal dampers produced comparatively high tip deflection reductions. The hybrid friction-VE dampers performed in a more stable and reliable manner than the diagonal and chevron brace dampers, but the resulting tip defelection reductions were slightly lower. The lower toggle VE damper displayed the highest performance and reliability from all damping systems.

A strategy for protecting buildings from earthquakes is to limit the tip deflection which provides an overall assessment of the seismic response of the structure. To this end, findings of the present study demonstrate that friction dampers are most effective when placed close to regions of the maximum interstorey drifts, whereas VE dampers are most effective when placed in the lowest storeys. The combined damping system, which consists of the diagonal friction damper placed in the storey with the highest interstorey drift and the diagonal VE damper placed in the lowest storey is clearly more effective than the hybrid friction-VE dampers; nevertheless the lower toggle VE damper seems to be the best choice for seismic mitigation.

Results from the study on the seismic response of the building facade system confirm the effectiveness of energy absorbing connections in the from of VE dampers to mitigate the adverse seismic effects on the response. In general, good seismic control was achieved for all cases with > 50% reductions in the response parameters. Control of differential displacement between frame and facade and inter-storey drifts were more effective in the upper storeys. The energy absorbing connections not only mitigated the façade failure, but also enabled significant structural control by reducing the inter-storey drifts. This study has domonstrated that it is possible to control facade distortions within acceptable limits and prevent failure. It has also demonstrated that energy absorbing connections are able to reduce inter-storey drifts and mitigate the detrimental seismic effects on the entire building facade system. The results presented in this paper were obtained for the chosen structures, but they may be useful in the seismic mitigation of other similar structures.

References

Abbas, H. and Kelly, J.M. (1993) A methodology for design of VE dampers in earthquake-resistant structures. *Technical report UCB/EERC-93/09*, Earthquake Engineering Research Center, University of California, Berkeley.

Constantinou, M.C., and Sigaher, A.N. (2000) Energy dissipation system configurations for improved performance. Proceeding of the 2000 Structures Congress and Exposition, ASCE, May 8-10, 2000, Philadelphia, PA.

Hanson, R., (1993) Supplemental damping for improved seismic performance. Earthquake Spectra, 9(3), 319-334.

Hareer, R., (2007) "Seismic Response of Building Façade System with Energy Absorbing Connections", PhD thesis, Queensland University of Technology, Brisbane, Australia.

Marko, J., Thambiratnam, D. P. and Perera, N. J. (2006) Study of viscoelastic and friction damper configurations in the seismic mitigation of medium-rise structures" Journal of Mechanics of Materials and Structures, October, pp. 1100-1151.

Welch, P.D, (1967) The use of Fast Fourier transform for the estimation of power spectra: a method based on time averaging over short, modified periodograms. IEEE Trans. Audio Electroacoust. Vol.AU-15 June, pp 70-73.