Effect of coupling between the moment frame and the viscous damper on seismic capacity of a steel building

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ABSTRACT

Viscous dampers can absorb energy and add damping to buildings to reduce their seismic responses. Damping frames may be external or internal to the seismic moment frame and may have no shared elements, some shared elements, or all elements in common with the moment frames. This study compares the seismic capacity of the steel structure considering the effect of coupling between the damping frame and the seismic moment resisting frame.

Nonlinear time history analysis of 10-storey steel moment frame structure with fully coupled and decoupled damping frames has been carried out under the number of ground motions. Plastic hinges considering axial-moment demand interaction have been considered for the moment frames.

The results of the analysis show that by fully coupling the damping frame to the seismic moment frame, the seismic capacity of the moment frame is decreased. This is because of having additional axial demands on the beams and columns from nonlinear viscous dampers will reduce the strength and plastic rotation capacity of the steel frames.

From the above comparison, it is realised that the architectural planning effectiveness of the dampers and the effect of coupling between the damping and moment frames should be considered for the design of seismic frames.

1 INTRODUCTION

The typical philosophy in the conventional seismic design is that a structure is permitted to undergo damage when subjected to a design level earthquake excitation. As a consequence, plastic hinges in the structure must be developed to dissipate the seismic energy. However, the development of the plastic hinges relies on large inelastic deformations to achieve a high level of structural ductility. The more ductility a structure sustains, the more structural damage it suffers.

Recently various types of supplemental damping devices have been developed around the world to minimize the possibility of structural and non-structural damage. Supplemental damping devices can absorb energy and add damping to buildings to reduce their seismic response. Among others, supplemental viscous dampers have gained increasing popularity in recent years attributing to their negligible influence on the fundamental natural period of the structure and their ability to provide this damping with negligible damage to the units themselves.

Viscous dampers have the unique ability to simultaneously reduce both stress and deflection within a structure subjected to a transient. This is because a viscous damper varies its force only with velocity, which provides a response that is inherently out-of-phase with stresses due to flexing of the structure.

Viscous dampers are classified into linear and nonlinear dampers based on their behaviour. In this linear model, the damping force is directly proportional to the velocity of the structure. As the structure moves faster, the damping force increases linearly. Nonlinear viscous dampers exhibit a damping force that is not directly proportional to velocity. The force-velocity relationship for a nonlinear viscous damper can exhibit saturation effects, with the damping force plateauing at high velocities.

Nonlinear fluid viscous dampers, with exponents in the range of 0.3 to 0.5 are often used in design for the following reasons: (a) force output increments are small at large velocities, therefore limiting the force output of the damper for velocities in excess of the design velocity (b) energy dissipation per cycle is more than that of comparable linear viscous dampers and the dampers are more effective for smaller magnitude earthquakes, and (c) such devices are suitable for attenuating the effects of high velocity pulses that may occur at near-fault locations (FEMA P-1051, 2016).

However, with nonlinear dampers you lose the benefit of the maximum velocity (dampers force) and maximum displacement (maximum elastic story shear) being nominally out of phase by 90 degrees. Based on ASCE 7-16 standard, for damper with exponent of 0.5, 0.7 factor is suggested for the concurrency of maximum response of the damper at the peak response of the moment frame.

Moreover, damping frames may be external or internal to the seismic moment frame and may have no shared elements, some shared elements, or all elements in common with the moment frames as shown in Figure 1. In the situation where the damping system shares the element with the structural resisting system, the dampers with larger force capacity may be required such that the size of structural members adjacent to the dampers may be a concern.

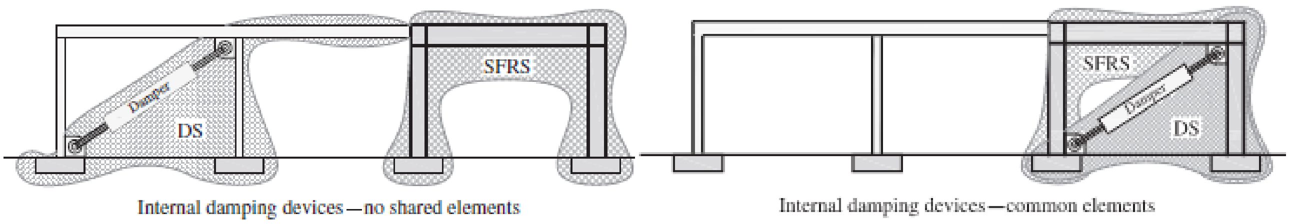


Figure 1: Damping System (DS) and Seismic Force-Resisting System (SFRS) Configurations (ASCE 7-16)

This study compares the seismic capacity of a 10-storey steel building considering the effect of coupling between the nonlinear damping system and the seismic moment resisting frame using response time history analysis.

2 CASE STUDY

2.1 Properties of the structure

The case study is a ten-story 2D steel framed office building with an Importance Level of 3 located in Wellington, NZ with a soil type D. The model configurations are shown in Figure 2. A constant mass of 100 Tons was lumped at each floor. The structure is also assumed to have constant story height, h, of 4m and bay length of 8m. The main period of the structure obtained from the dynamic analysis was 1.2s which has been used in scaling the ground motions to the target spectrum. The main frame without viscous damper is designed to 75% of the total base shear based on ASCE 7-16 to have complete load path and resilient structure with ductility of 2.0 and Sp of 0.7. Nonlinear plastic hinges have been considered based on ASCE 41-17 considering axial-moment demand interaction as shown in Figure 3.

(a) (b)

Figure 2: Case Study Model

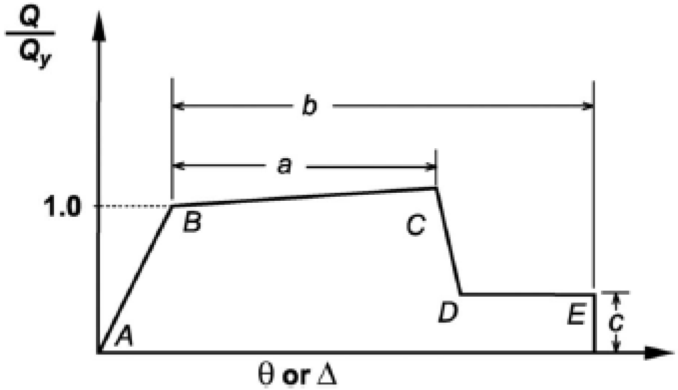


Figure 3: Generalized Force-Deformation Relation for Steel Elements

3 DESIGN METHODS

3.1 Shear Strain Energy Method

Based on this method, the damping coefficient along the height of the structure is distributed with respect to the storey shear strain energy corresponding to the first vibration mode of the structure in the direction of consideration. This concept has been adopted in some seismic design codes such as ASCE 41-17 and ASCE

7-16. The storey strain energy in the frame (U_i) is determined using Eq.1 where F_i and δ_i shall be taken as the inertia force and floor displacement at floor level i.

$$U_i = \frac{1}{2} F_i \times \delta_i \tag{1}$$

This method is considered rational because putting more damping coefficient at the location where the storey shear strain energy is larger will result in a greater contribution from the viscous dampers to the system damping ratio such that the dampers can be used more efficiently.

The damping constants (C_L) for linear viscous dampers shall be determined using Eq.2 where β_v is amount of viscous damping and ϕ_{i1}, is the fundamental mode shape of the structure:

$$\beta_v = \frac{E_{vd}}{4\pi E_{es}} = \frac{\pi \sum_1^{N_d} c_{Li}\varphi_{i1}^2 cos^2\gamma_i}{T_1 \sum_1^{N_f} k_i\varphi_{i1}^2} \tag{2}$$

The damping constant for a nonlinear viscous damper is determined using Eq.3 (Christopoulos and Filiatrault, 2006) where α is the damping exponent (between 0.1 to 1.0), ω is circular frequency of the structure and X_0 is the design displacement of the dampers:

$$C_{NL} = C_L \frac{\sqrt{\pi}}{2} (\omega X_0)^{1-\alpha} \tag{3}$$

Table 1 shows the designed damper coefficients over the height of the building. It is seen that the largest damping coefficient from SSE is located at the first storey. At some storeys especially higher storeys where the storey shear strain energy is small, the damping coefficients are small. Table 1 also shows the maximum damper forces under Northridge-01 ground motions along the height of the building.

Table 1: Damper coefficients and forces (Northridge-01)

Storey	**SSE**	
	C (kN-s/m)	F (kN)
10	980	465
9	1060	510
8	1156	533
7	1880	1086
6	1910	1116
5	2010	1129
4	2135	1097
3	2900	1580
2	3000	1500

1	3101	1230

4 GROUND MOTIONS SELECTION AND SCALING

Three ground motions have been used for analysis of the structure as shown in Table 2. For the purposes of the comparative study the building was modelled in 2D using ETABS 2018. Nonlinear response time history analyses were performed on the prototype building with non-linear elements used for the viscous dampers, and plastic hinges of moment frames. Railey damping with proportional damping of 5% is considered. All earthquake records are scaled to the period range of $0.4T_1$ to $1.3T_1$. T_1 is the longest translation period of the building.

Table 2. Input earthquakes

RSN	Earthquake	R_{JB} (km)	Mw	Scale factor (MCE)
1085	Northridge-01	0.0	6.69	1.2
1503	Chi-Chi, Taiwan	0.6	7.62	1.7
4040371	Tohoku, Japan	22.8	9.12	2.5

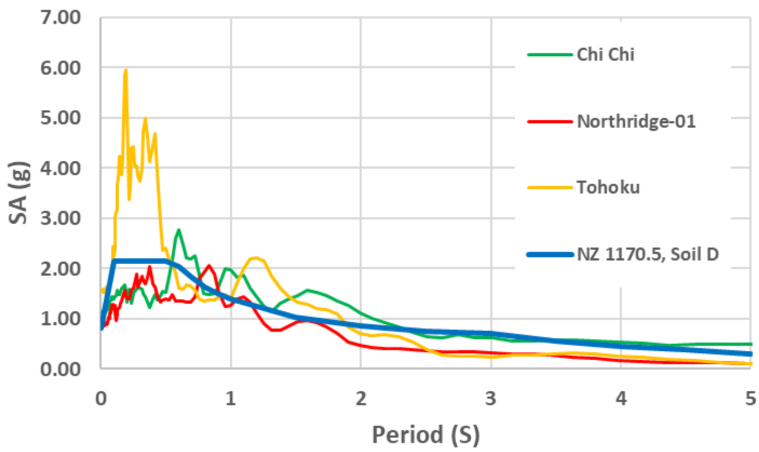


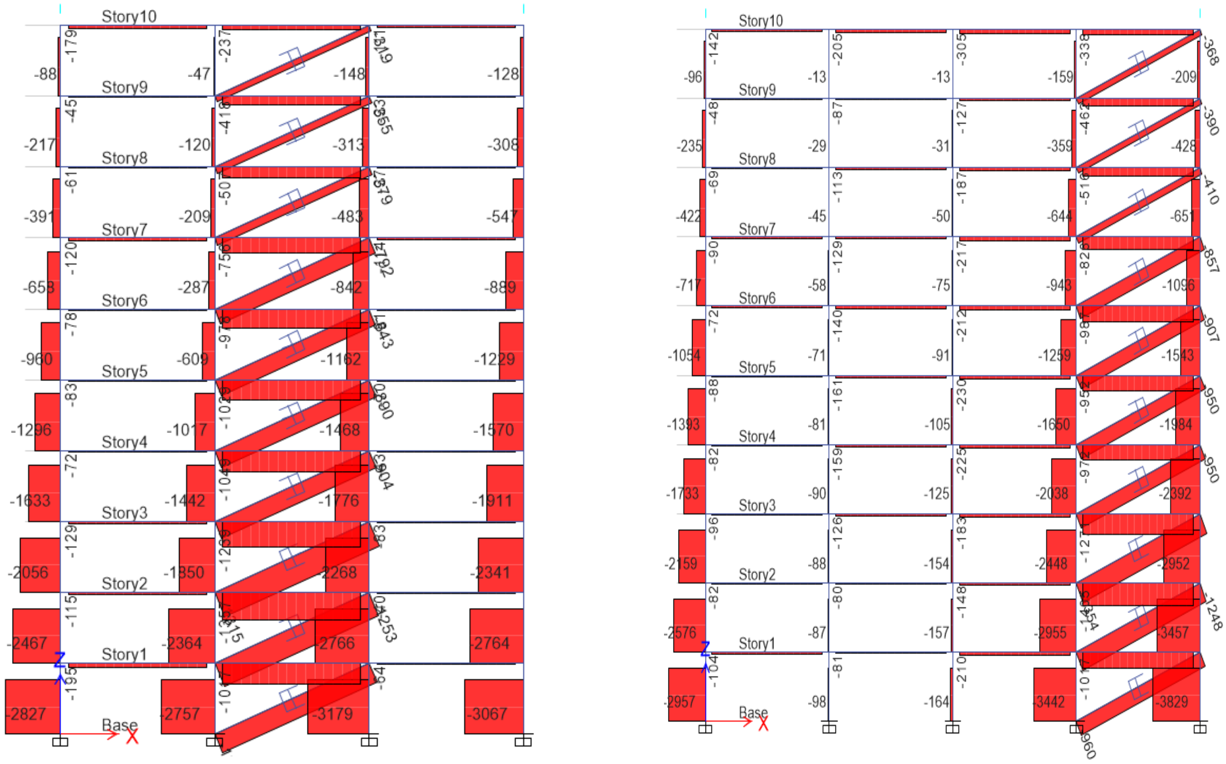
Figure 4: Spectral Acceleration of Ground Motions vs NZ1170.5 (Soil D, MCE)

5 RESULT OF ANALYSIS

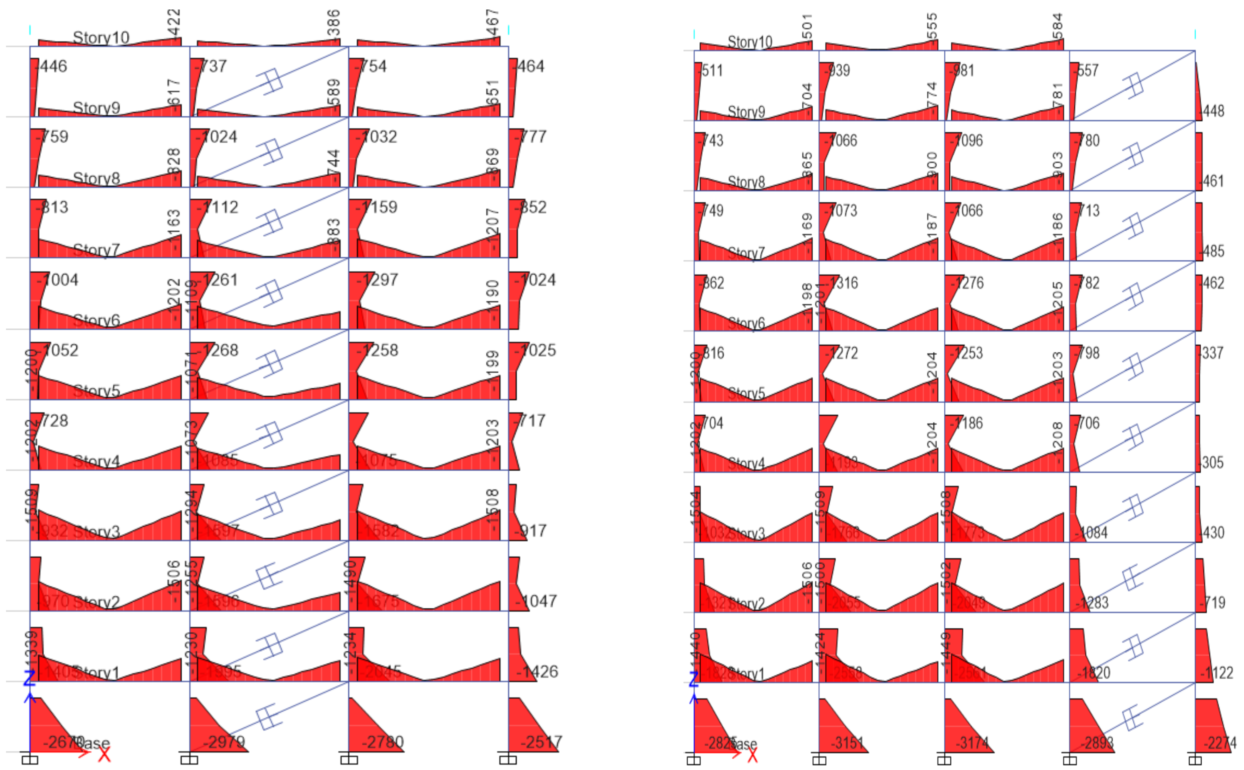
Figure 5 compares the maximum compression and moment demands on the structural members adjacent to the dampers in two different configurations. It shows that the beams in first configuration where acting as both moment frame and collector element goes under compression and moment demand. However, in the second configuration, the beams only act as collector elements and transfer the load to the seismic moment frames. Based on that the structural members adjacent to the larger damper forces shall have sufficient capacity to transfer the larger forces.

Figure 6 shows the plastic hinges status of the building with two different configurations; i) with common elements and ii) some shared elements between the damper system and MRF system. It shows having the common elements, the capacity of the frames decreases, and the number of the elements have D (purple colour) or E (red colour) plastic rotation status based on Figure 3. This is only happened in the middle bay where there is high axial loads from the viscous damper and this reduce the plastic rotation capacity of the beam by 4 times for $0.2<N^*/N_{cr}<0.5$ based on the ASCE 41-16 and the strength capacity of the steel beams are also decreased by considering axial-moment interactions.

Figure 7 also compares the plastic rotation of the beam for both configuration under Northridge-01 record. It shows with common element configuration, the beam has less flexural strength capacity and it goes under huge rotation. Because of that its shows with red colour status in Figure 6.

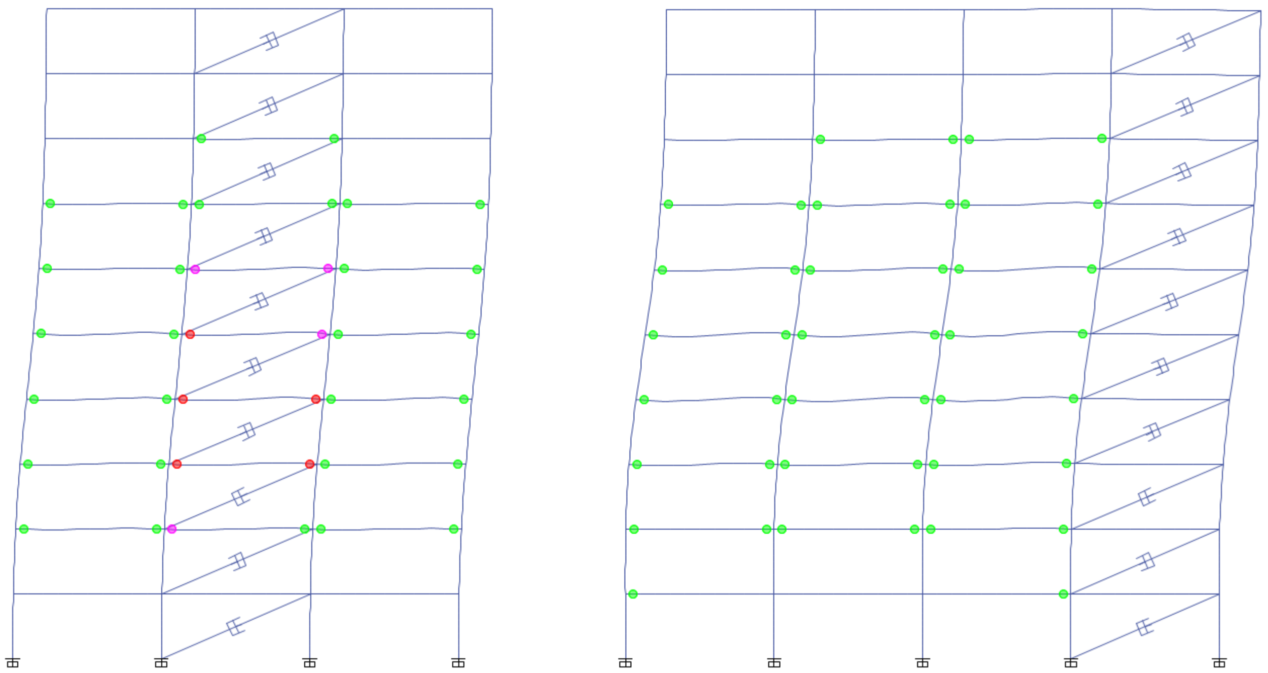


a) Maximum Compression Demand

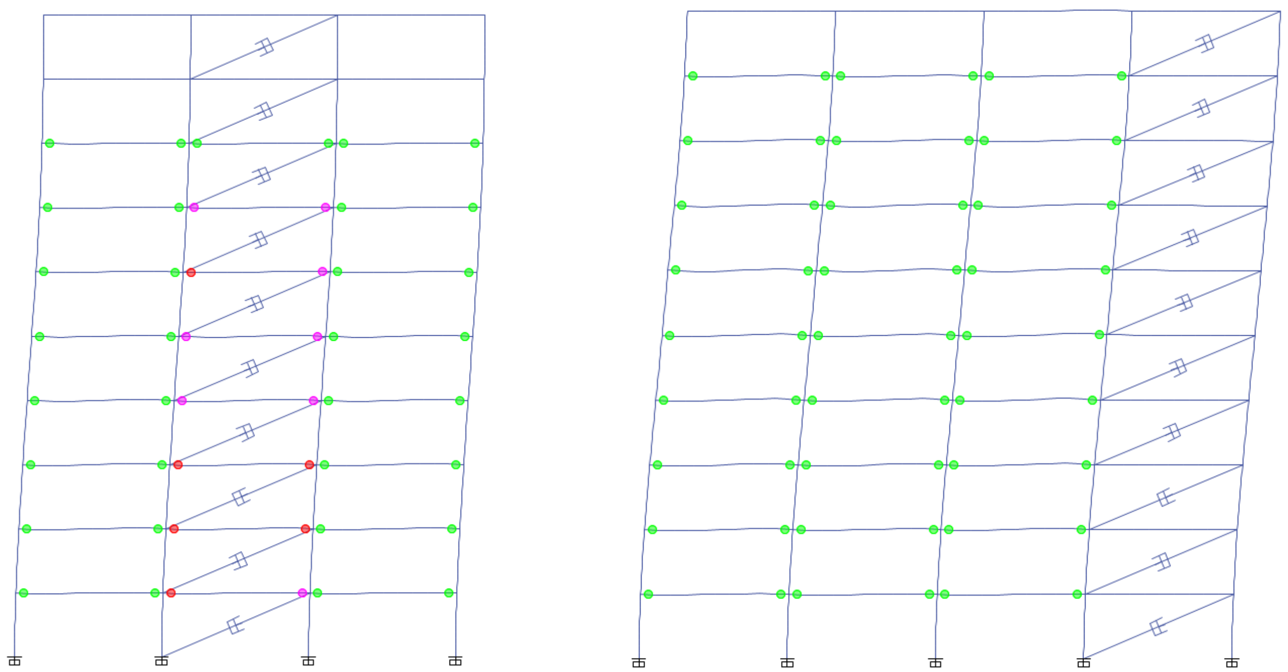


b) Maximum Moment Demand

Figure 5: Comparison of compression and moment demands on structural members adjacent to viscous dampers under Northridge-01 ground motion.

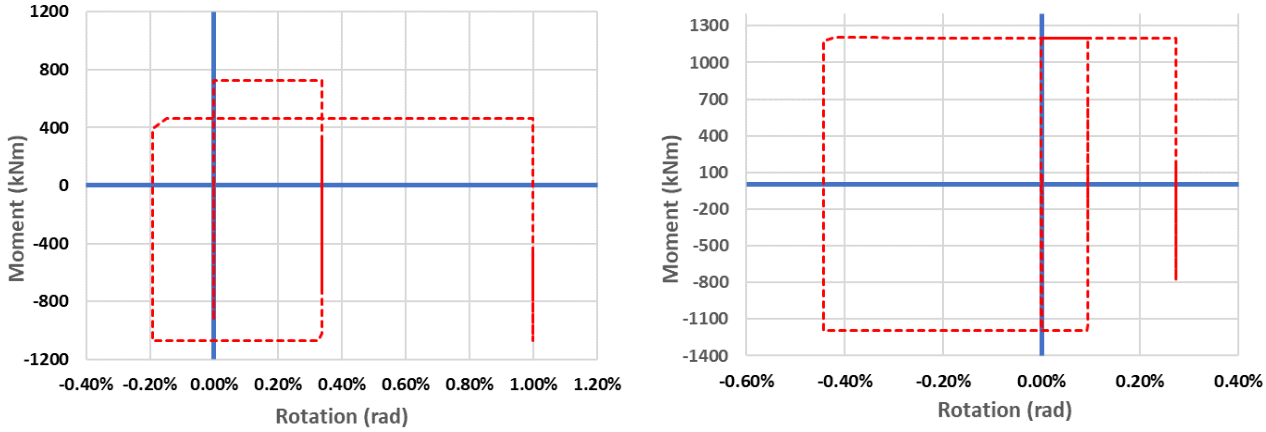


a) Northridge-01



b) Tohoku, Japan

Figure 6: Plastic Hinges Status the structure at the end of the time history analysis.



i) With Common Elements ii) With Some Shared Elements

Figure 7: Moment-Rotation behaviour of the Level 4 middle Beam under Northridge-01 record.

6 CONCLUSION

Based on the response history analyses conducted on the prototype buildings using the real ground motions, the following conclusions can be made:

1. Using nonlinear dampers, the benefit of the maximum velocity (dampers force) and maximum displacement (maximum elastic story shear) being nominally out of phase will be lost.
2. The structural members adjacent to the larger damper forces shall have sufficient capacity to transfer the larger forces. The additional axial demands on the beams and columns from nonlinear viscous dampers should be considered in design.
3. By fully coupling the damping frame to the seismic moment frame, the moment and rotation capacity of the seismic moment frame is decreased.
4. The placement of the damper relative to the moment frame actions is critical and can amplify the axial loads on the beams and columns.

7 REFERENCES

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