

# Analytical Investigation of the Effects of HF2V Damping Devices on the Seismic Performance of the SAC3 Building

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## Abstract

In this paper, the seismic performance of the 3 storey seismic frame with rigid moment connections at the beam ends (commonly known as the SAC3 building) is compared with that of the same frame using semi-rigid connections with high force to volume (HF2V) lead dissipators. The presence of the gravity frames in the model is also considered. It was found that the placement of dissipators, ignoring the effect of the gravity frames, caused a 12% increase in period of the frame due to the decreased stiffness of the connections. During design level ground shaking the semi-rigid connections with HF2V dissipators have slightly lower accelerations, up to an 80% increase in peak drift, and a 200% increase in the permanent displacement compared to the as-designed case, but no structural damage is expected. When the gravity frames are considered, the floor accelerations decrease further, the peak displacements do not significantly change, but the residual storey drift ratios reduce to approximately 0.17%. This result is less than one half that of the as-designed frame, where typically gravity frame effects are not considered. The analyses show that combining HF2V lead dissipators with gravity frames and well-designed non-structural elements creates a system with almost no structural damage and low residual displacements.

**Key words:** HF2V, Dampers, Seismic Response, Finite Element Analysis, FEA

## 1. Introduction:

Steel moment frame structures exposed to moderate or strong ground motions are designed to accept damage in the beam end plastic hinge zones or in the beam-column joint panel zones to dissipate seismic energy. Large permanent displacements may be present at the end of the earthquake shaking. Repair costs for such damage and consequent downtime can be substantial, creating significant economic and business impact.

The use of a Damage Avoidance Design (DAD) approach, with relatively damage-free connections, can reduce repairs and minimize disruption, which would substantially reduce economic and business impacts. Damage avoidance may be achieved in moment frames by using specialised devices whose dissipative performance does not degrade on subsequent cycles of use. Ideally, such devices would perform in a consistent and repeatable manner on every response cycle throughout the life of a structure, and would not require maintenance or replacement after a seismic event.

Energy dissipation devices using lead were proposed by Robinson and Greenbank [1] to absorb energy in a controlled, repeatable manner as a base isolation system. Lead is ideal

for this purpose due to its unique rheological properties, low re-crystallisation temperature, and ability to allow any residual compression forces in the device to creep back towards zero over time [2, 3]. A summary of the state of the practice developed from this early work is given by Cousins and Porritt [4]. While these devices were ideal for their intended purpose [5], they were too large to fit within standard structural connections, despite having the necessary resisting force and energy dissipation capacity.

Recently developed high force-to-volume (HF2V) dissipation devices of compact size [2] can fit directly into beam-column connections [2, 3, 6, 7] to enable true damage avoidance connections. The device consists of a steel cylinder filled with lead and through which a shaft with a bulge passes. As the shaft is moved, the lead moves around the bulge providing a resisting force. These devices may be modeled as weakly velocity sensitive non-linear viscous dampers, where higher velocities yield a slightly greater resisting force. Relatively inexpensive to manufacture, they have been experimentally characterized with full details [2, 3]. Energy is dissipated on every dynamic response cycle without strength degradation or damage [8], in contrast to conventional steel connections or connections with sacrificial dissipators [9, 10].

To address steel moment frames, it is necessary to answer the following questions:

- 1) Can HF2V devices be realistically used in steel frames?
- 2) Can these devices in steel joints be modelled appropriately?
- 3) Are there ways to minimize permanent displacements of the structure?
- 4) How does the response change when using these devices, instead of conventional connections, in modern steel moment resisting frames?

This paper conducts several non-linear analyses to compare the seismic performance of the SAC Steel Project 3-story structure with and without the HF2V devices.

## 2. Applying HF2V Devices to Steel Frame Structures:

A simple device model with low computational demand is required to analyse these devices in large non-linear dynamic simulations. The HF2V joint connection design is modeled as a modified rigid connection with a pin based on experimental results that showed damage free performance for interstory drifts out to 4% [6]. Similar results were obtained in DAD concrete connections [7, 11]. In this research, a rotational hinge element, containing elastic, plastic and velocity dependent (damping) aspects, was developed to match experimental results and used in ABAQUS. A comparison between experimental data and this model developed is shown in Figure 1. The moderate differences at large story drifts are attributed to experimental setup and experimental control issues [6] and not to specific device behaviour. The model is thus sufficiently reliable for large structural system analyses.

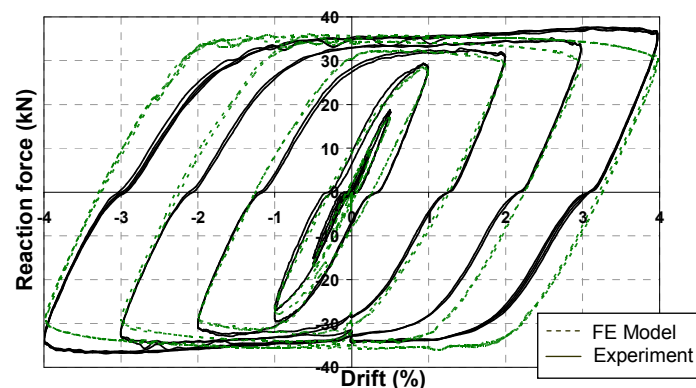


Figure 1: Finite element model and experimental results for HF2V steel connection.

The HF2V devices exhibit behaviour similar to that from an elastic perfectly plastic (EPP) structure as shown in Figure 1. Placed in a real structure, P-delta effects on the frame are likely to cause the curve to have a negative post-elastic stiffness ratio,  $r$ , where  $r$  is shown schematically in Figure 2a. Single-degree-of-freedom (SDOF) oscillators with  $r \leq 0$  have little inherent dynamic re-centering capability, as shown by the average residual displacement ratio,  $\Delta_r/\Delta_{r,max}$ , in Figure 2b, where  $\Delta_r$  is the residual displacement and  $\Delta_{r,max}$  is the maximum possible residual displacement for the specified target ductility [12]. Because of the hysteresis curve shape, even if no structural damage occurs, the structure may have large permanent displacements making it vulnerable to aftershocks and difficult to straighten after an earthquake.

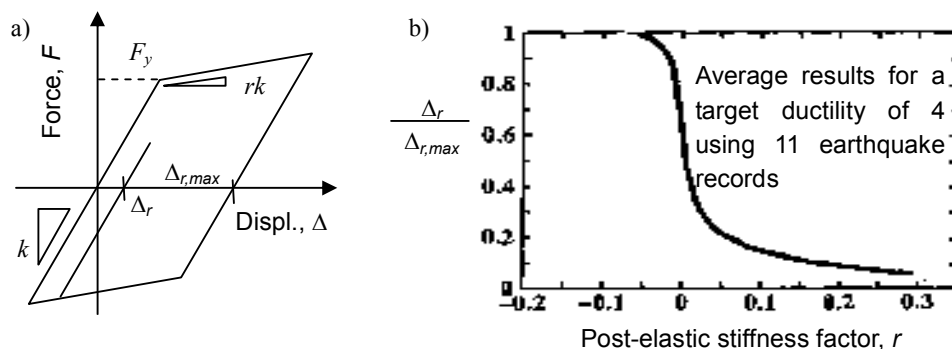


Figure 2: Effect of Hysteresis Curve on Residual Displacement Ratio: a) Schematic Hysteresis curve, and b) Average Residual Displacement Ratio [13].

Figure 2 shows that one means of improving response is to increase the post-elastic stiffness factor,  $r$ , until it is significantly positive. Another method in multi-storey frames is to consider the effect of continuous columns over the height of the structure. Such columns, which include the seismic in-plane columns, the seismic out-of-plane columns and all gravity columns, provide continuity between stories. Thus, the response of the overall frame is not like that of the SDOF oscillators in Figure 2. In fact, MacRae et al. [14] have shown that continuous columns reduce the drift concentrations in individual stories. In many dynamic analyses, the effect of column continuity is often not considered, except in the seismic frame analyzed, and gravity frame effects are thus often ignored. Based on these results, large permanent displacements may be mitigated by either increasing the post-elastic stiffness of each story, or by providing/considering column continuity.

While post-elastic stiffness of the HF2V device is close to zero, the post-elastic stiffness of the total storey in a steel frame may be greater than zero, primarily as a result of the rotational stiffness of the gravity beam end connections. This added stiffness contribution, which can be controlled to some extent in the design and is in addition to the contribution of column continuity, is investigated in this research.

### 3. Frames Analysed:

The structural system used in this investigation was developed as part of the Los Angeles SAC Steel Project [15, 16]. The structure and earthquake suite used in this research were developed for the Los Angeles area [17]. The specific structure in this study is the three-storey steel building designed for Los Angeles, also called SAC-3 or LA-3, with moment resisting frames only at the periphery [16]. Each bay has centerline dimensions of 9.14 m by 9.14 m and the columns extend over the 3 stories of 3.96m height each. The structure is nearly uniform in the two orthogonal directions. The horizontal seismic weight per frame at levels 3, 2 and 1 are 5200kN, 4800kN and 4800kN respectively.

For the 2-D analyses here, only the east half of the building is modeled. The seismic

frame is modeled directly, but to further reduce the total number of degrees of freedom, the other columns on the east half of the building are merged into a single "consolidated gravity column" (Axis E) by summing the stiffness and strength of the individual columns considering deformation in the N-S direction. The consolidated gravity column is slaved in the horizontal direction at every floor to the seismic frame to form a complete two-dimensional model shown in Figure 3. Pins at the ends of the beams in the right hand bay represent perfectly pinned connections. With these pinned connections, the bay width in the right hand bay is unimportant. However, this width was assumed to be one half of the actual bay width for convenience in different case studies, as discussed later.

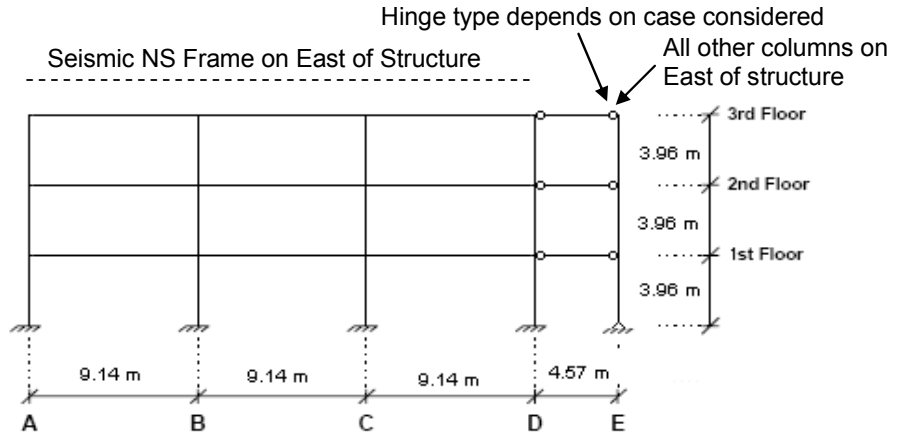


Figure 3: Model 1 - As-Designed SAC3 structure.

The second and third models considered, Model 2 and Model 3, used HF2V devices in the seismic frame, rather than the as-designed rigid beam-column connections. These modified connections are shown as rectangular boxes between grid lines A – D of Figure 4. As a result, it is expected that this change in connection design will affect the structural stiffness and natural period, as well as its ability to dissipate energy.

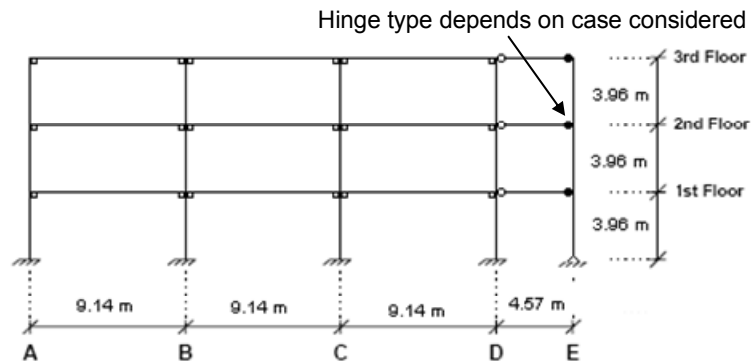


Figure 4: Models 2 and 3 - With HF2V devices in the seismic Frame.

The HF2V device force capacity is chosen to achieve the base shear strength required by FEMA 450 [18]. The seismic base shear was determined by Equation (1) where  $C_s$  is the seismic coefficient and  $W$  is the structure's seismic weight.

$$V = C_s W \tag{1}$$

where  $C_s$  was determined by Equation (2),  $S_{DS}$  = the design spectral response acceleration parameter,  $R$  = the response modification factor and  $I$  = the occupancy importance factor.

$$C_s = S_{DS} / (R / I) \quad (2)$$

For the Los Angeles area, soil type  $D$ , an  $R$  factor of 8, and a period,  $T$ , of 1.0s, means  $C_s = 0.05$ . Hence, the design strength for this structure is 5% of its seismic weight. Model 2 was provided with devices that allowed this base shear value to be reached.

Due to the possibility of over-strength, and to have a more reasonable comparison with Model 1, Model 3 was designed to resist twice this base shear, or 10% of the total weight. In both Models 2 and 3, HF2V device capacities were thus selected so that shear resistances at each level over the seismic frame height were proportional to the weight above.

For each of the seismic frame models, Models 1-3, three separate cases for the gravity beam end connection rotational stiffness and strength were considered. Parameters describing the bilinear beam end rotation stiffness and strength values were provided beside Column E. These parameters are given in Table 1.

- **Case 1** describes a major effect of the gravity beam (and slab) end connection, where the rotational stiffness,  $k = 2.5EI/L$  and the strength,  $M_s$ , is 100% of the plastic moment of the beam,  $M_p$ .
- **Case 2** describes much less effective connections with  $k = 1.7EI/L$  and  $M_s$  of 50% of  $M_p$ .
- **Case 3** represents the case of a perfect pin with no rotational stiffness ( $k = 0$ ).

The values chosen in Cases 1 and 2 are based on previously published experimental studies [19, 20]. In the span D-E beam, the point of inflection in the gravity beams is 4.57m from Column E, which is the expected location of the point of inflection due to seismic forces. This point is shown by the white circles representing perfect pins on the right of column D in Figures 3 and 4.

Table 1: Analysis matrix and model/device properties for each case.

Model	Model Characteristics	Case	Gravity Beam End Connection for Each Case
1	As- designed seismic frame	1	$k = 0$
		2	$k = 1.7EI/L, M_s = 50\%M_p$
		3	$k = 2.5EI/L, M_s = 100\%M_p$
2	With HF2V Devices - Design Base Shear is 5% of Weight	1	$k = 0$
		2	$k = 1.7EI/L, M_s = 50\%M_p$
		3	$k = 2.5EI/L, M_s = 100\%M_p$
3	With HF2V Devices - Design Base Shear is 5% of Weight	1	$k = 0$
		2	$k = 1.7EI/L, M_s = 50\%M_p$
		3	$k = 2.5EI/L, M_s = 100\%M_p$

The different models (representing the seismic frame characteristics) and the different cases (representing the gravity beam end connection bilinear parameters) are given in Table 1. Model 1 Case 1 (M1C1) represents the SAC LA3 building design as it is generally analyzed with no extra consideration for gravity frame effects. The fundamental periods varied from 1.0s for M1C1 to 1.12s for M3C1. Initial stiffness proportional Rayleigh damping of 5% was used in the first mode. Velocity dependence of the device dissipation was also incorporated, as reported by Rodgers et al [2, 3].

Dynamic inelastic time history analyses were conducted with ABAQUS using the LA medium suite of the earthquake records from the SAC Steel Project [17]. These 20 earthquake records have a magnitude between 6.5 - 7.25, epicentral distances of 5 - 40

kilometers and an exceedance probability of 10% in 50 years. They are thus design basis earthquakes (DBE).

#### 4. Results and Discussion:

The total floor accelerations, relative residual roof drifts and relative storey drifts are listed in Tables 2 to 4. In each case, the median (50th percentile) and appropriate lognormal standard deviation [21] are calculated based on the 20 data points, one from each ground motion response in the earthquake suite used.

Table 2: Response for Gravity Beam End Connection Case 1 ( $k=0$ ).

	Model 1		Model 2			Model 3		
	As built		5 % Baseshear			10 % Baseshear		
	lognormal mean	deviation	lognormal mean	deviation	change [%]	lognormal mean	deviation	change [%]
Peak Acc Floor 1	8.062	1.640	7.580	1.597	-6.362	7.423	1.618	-8.613
Peak Acc Floor 2	6.892	1.535	5.437	1.668	-26.767	5.412	1.665	-27.341
Peak Acc Floor 3	8.250	1.483	5.772	1.534	-42.938	5.626	1.535	-46.636
Relative Roof Drift	0.185	1.916	0.359	1.430	94.295	0.327	1.413	76.582
Residual Roof Drift	0.039	2.657	0.120	2.366	207.088	0.067	5.787	70.848
Interstorey Drift 3-2	0.075	1.353	0.133	1.380	77.848	0.122	1.357	63.188
Interstorey Drift 2-1	0.070	1.383	0.122	1.435	73.323	0.111	1.411	58.646
Interstorey Drift 1-0	0.068	1.357	0.110	1.444	60.480	0.089	1.684	30.430

Table 3: Response for Gravity Beam End Connection Case 2 ( $k=1.7EI/L$ ,  $M_{pl}=50\%$ ).

	Model 1		Model 2			Model 3		
	As built		5 % Baseshear			10 % Baseshear		
	lognormal mean	deviation	lognormal mean	deviation	change [%]	lognormal mean	deviation	change [%]
Peak Acc Floor 1	7.874	1.665	7.379	1.624	-6.716	7.228	1.643	-8.950
Peak Acc Floor 2	6.837	1.526	5.689	1.653	-20.169	5.582	1.610	-22.475
Peak Acc Floor 3	8.645	1.447	6.094	1.534	-41.866	5.940	1.511	-45.532
Relative Roof Drift	0.194	1.370	0.306	1.380	57.301	0.280	1.387	44.105
Residual Roof Drift	0.015	4.408	0.022	2.934	40.491	0.020	2.273	30.161
Interstorey Drift 3-2	0.069	1.353	0.114	1.289	65.711	0.104	1.310	51.940
Interstorey Drift 2-1	0.066	1.367	0.104	1.378	58.551	0.095	1.384	45.344
Interstorey Drift 1-0	0.067	1.350	0.097	1.342	45.064	0.088	1.359	32.319

Table 4: Response for Gravity Beam End Connection Case 3 ( $k=2.5EI/L$ ,  $M_{pl}=100\%$ ).

	Model 1		Model 2			Model 3		
	As built		5 % Baseshear			10 % Baseshear		
	lognormal mean	deviation	lognormal mean	deviation	change [%]	lognormal mean	deviation	change [%]
Peak Acc Floor 1	7.841	1.525	7.315	1.633	-7.192	7.170	1.609	-9.357
Peak Acc Floor 2	6.791	1.434	5.722	1.634	-18.689	5.595	1.620	-21.377
Peak Acc Floor 3	8.738	1.480	6.071	1.543	-43.923	5.953	1.510	-46.776
Relative Roof Drift	0.193	1.367	0.296	1.383	53.576	0.272	1.399	41.343
Residual Roof Drift	0.015	4.832	0.021	3.306	43.479	0.019	3.444	29.323
Interstorey Drift 3-2	0.067	1.362	0.110	1.291	65.111	0.101	1.319	51.888
Interstorey Drift 2-1	0.065	1.362	0.101	1.380	56.059	0.093	1.395	43.738
Interstorey Drift 1-0	0.066	1.352	0.094	1.341	41.970	0.086	1.373	30.037

As seen in Tables 2-4, the total floor accelerations were between 6% and 46% less for the structures with HF2V devices (Models 2 and 3) compared to the as-designed structure (Model 1). The greatest reduction occurred in the upper stories. Accelerations for the 5% and 10% base shear cases (Models 2 and 3) were similar, as were the lognormal standard deviations, indicating no change in potential damage to occupants and contents. However, overall the reductions seen with the devices are significant and reflect significantly reduced damage and thus costs or injury.

The similar lognormal deviations indicate that the HF2V design did not change the distribution of the responses, but merely shifted them to lower values. As noted, the reductions in total storey accelerations should significantly increase occupant and contents

safety. The bigger damping and the increase in period are reasons for reduced accelerations in the stronger frame.

The median increase in peak storey drift is between 30% and 78%. This result can be attributed to the increased period due to the change made to incorporate the device by weakening selective connections to allow their free movement and use. This increase is greatest in Model 2 (with 5% base shear) for Case 1 where gravity frame effects are ignored because there is no added recentering stiffness to resist the motion. Note that no damage to structural connections is expected with the HF2V devices in the seismic frame (Models 2-3). Nevertheless, the increase in storey drifts may incur damage to poorly detailed non-structural elements, such as cladding and internal partitions. However, if the non-structural elements are designed and detailed to sustain these larger (but not unrealistic) drifts without damage, then no non-structural damage is expected.

The median relative roof drifts are increased by 35% to 94%, which is similar to the increase in drifts. Figure 5 shows the relative roof displacement for the Loma Prieta earthquake record in this medium suite of ground motions and compares the Model 1 Case 2 response to that of the Model 2 (5% base shear) Case 2 response. It can be clearly seen that the peak displacement is significantly higher for the 5% base shear design structure.

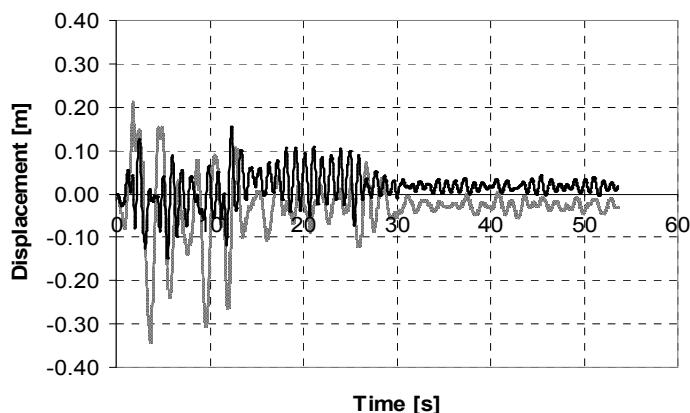


Figure 5: Relative Roof Displacement History for LA01 and  $k=1.7EI/L$ .

The presence of the HF2V devices (Models 2 and 3) increased the median relative roof residual (or permanent) displacement by almost 100% for the Case 1 frame (with  $k = 0$  at all gravity beam ends). However, for the frames with the gravity beam end connection stiffness and strength considered, the median increase was less than 50%.

Even though the peak displacements of the models containing dissipators, Models 2 and 3, are more than twice that of the as-designed (Model 1, i.e.  $k=0$ ) structure, the presence of the gravity frame stiffness and strength (i.e. Cases 2 and 3) decreases the residual displacement to about one half of that of the as-designed ( $k=0$ ) structure. The median value of the relative residual roof drifts for the frames with devices when considering the gravity frames was less than 0.22%. This result is close to the allowable construction tolerance of 0.2%. Hence, designing recentering stiffness into gravity frame connections enables reduced permanent displacement despite damage-free increased transient response.

The distribution of residual drifts is much tighter for the structures considering gravity columns and their recentering stiffness. For example, the 95th percentile residual drift for Models 1, 2 and 3 were  $0.039m * 2.6572 = 0.275m$ ,  $0.671m$ , and  $2.243m$  for the Case 1 ( $k = 0$ ) frame, but it is  $0.291m$ ,  $0.189m$  and  $0.103m$  for the Case 2 frames. The combination of HF2V dissipators with the gravity frame stiffness is therefore very effective and a higher level of confidence can be ascertained to the performance of structures designed with these devices.



## 5. Conclusions:

This paper has explored the advantages of using HF2V energy dissipators at the beam column joints of steel moment resisting frames by analysing the SAC Los Angeles 3 storey seismic frame subject to the SAC Los Angeles medium suite records. It has incorporated re-centering stiffness both from the gravity columns and from the beam connection to gravity columns together with DAD connections using HF2V devices. The main results of this extensive series of nonlinear finite element analyses can be summarised:

- i) Placing HF2V lead dissipators in the seismic frame joints reduces the joint stiffness and increases the fundamental period, resulting in decreased floor accelerations, increased peak displacements and slightly increased residual displacements with respect to those obtained from the as-designed rigid jointed frame. Nevertheless, for frames with well-designed and separated non-structural elements, no damage is expected because all energy dissipation and non-linearity occurs in the damage free HF2V devices.
- ii) Gravity frames provide increased re-centering stiffness. When these were considered in the model, floor accelerations decreased further, and peak displacements decreased but were still greater than that of the as-designed structure. Most significantly, roof residual displacements reduced to approximately 50% of the as-designed structure not considering gravity frame effects.
- iii) The combination of HF2V dissipators with gravity frames and creates a system with lower floor acceleration and almost no damage despite slightly increased displacements. If drift-sensitive non-structural components are carefully designed, this system appears significantly superior to conventional construction methods, particularly with respect to the resulting economic, financial and business impacts.

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