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A 2-STEP P-SPECTRA-BASED RETROFIT METHOD FOR VERTICALLY IRREGULAR FRAMES USING SUPPLEMENTAL DAMPERS

Jack WW Guo
PhD Candidate, University of Toronto, Canada
wen.guo@mail.utoronto.ca

Constantin Christopoulos
Professor, University of Toronto, Canada
c.christopoulos@utoronto.ca

ABSTRACT: A direct performance-based seismic retrofit procedure for vertically irregular structures using combinations of passive supplemental damping is investigated in this paper. The proposed two-step procedure first uses hysteretic dampers to correct the vertical irregularity and then add supplemental damping in accordance with the performance spectra-based method, which allows designers to target global response quantities using an idealized SDOF system. Numerical validation of the proposed procedure is presented using idealized shear buildings. It is shown that this approach can effectively target performance goals. In particular, the use of viscoelastic dampers with small stiffness can help maintain the uniformity of the retrofit solution even after yielding occurs. On the other hand, it is found that for frames with a large strength deficiency, very large increase in base shear force can result from the direct application of this procedure, which may render the retrofit impractical.

1. Introduction

The performance-based seismic retrofit of inelastic vertically irregular structures can be a challenging task because these structures have a tendency to form a local mechanism, which significantly increases the inelastic demand in the vicinity, and make the prediction of their behaviour very difficult. While the seismic retrofit of vertically irregular structures is routinely done by traditional stiffening and strengthening, it is well recognized that such interventions can very well produce much larger seismic demands and can end up being inefficient and costly. On the other hand, more economical solutions may be achieved by using supplemental damping in conjunction with seismic stiffening and strengthening or selective weakening (Reinhorn et al. 2002, Viti et al. 2006, Kam and Pampanin 2008). A further extension to this type of hybrid intervention is to use only supplemental dampers to mitigate both the global seismic response and the irregularity in the structure in order to take advantage of the better quality control and easier installation of damping devices.

The recently proposed performance spectra-based methodology for the performance-based retrofit of structures using supplemental dampers can be modified to accommodate for vertical irregularities (Guo and Christopoulos 2012). However, having large vertical irregularity negatively impacts the accuracy of the P-Spectra method and limits its range of application since the altered distribution of supplemental dampers to account for vertical irregularities often counters the effectiveness of their energy dissipation function. Kyriakopoulos (2013) has use the performance spectra method to design supplemental damping retrofit for a 9-storey steel hospital structure in Quebec, Canada. It was found that due to the severe soft and weak storey, the local response of the retrofitted structure around the soft storey is larger than the performance target. In some cases, the local irregularity in the structure significantly altered its post-yield dynamic properties in such a way that supplemental dampers designed using one set of assumed inelastic mode shapes are no long able to achieve the target responses reliably. It is worth noting that P-Δ effects can amplify this problem due to the negative post-yield stiffness the structure experiences after yielding occurs.

In the case of viscoelastic dampers, the amplified permanent stiffness assigned to a severely irregular structure to correct local irregularities may actually introduce a new irregularity in the structure (Guo and Christopoulos 2011). These observations suggest that a more robust procedure for designing supplemental damping retrofits for vertically irregular structures is needed to provide a better balance between the role of energy dissipation and stiffness and strength correction. Hence, this work presents a numerical study to investigate a 2-step direct procedure that separates the irregularity correction from the design of the supplemental damping.

2. P-Spectra-based 2-step direct design procedure for vertically irregular frames

The first step in the proposed procedure utilizes the method described in Guo and Christopoulos (2011) to compute an approximate distribution of base frame properties that corrects the vertical irregularity in accordance to a predetermined irregularity criterion. Hysteretic dampers are then assigned to the original building to supplement the stiffness and strength deemed necessary to make the structure regular. The structure resulting from step 1 is herein referred to as the target base frame. It is an intermediate structure with added stiffness and strength (using hysteretic dampers), which is later treated as a regular seismic force resistant system.

Consider the frame structure shown in Fig 1. Using the seismic mass and elastic mode shape obtained from an eigenvalue analysis of the actual frame structure, an equivalent shear building can be constructed. Based on the stiffness and strength properties of the shear building, step 1 of the proposed procedure first distributes additional strength and stiffness to correct the irregularity. In step 2, supplemental damping is added to the corrected structure as if it is vertically regular in order to control the seismic response.

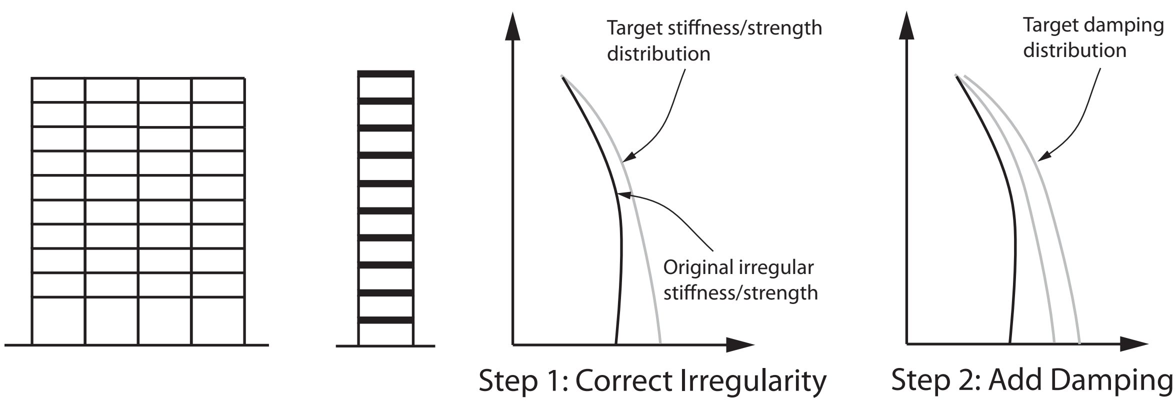

Fig. 1 – Schematic for the 2-step direct retrofit for vertically irregular structures

Given the storey masses, m_i, the values of the storey stiffness, $K_{f,i}$ and strength, $V_{f,i}$ of the equivalent structure can be explicitly expressed as a function of the fundamental period, T_1 and the mode shape, ϕ_i, if these quantities are assumed to be proportional as suggested by Priestly et al (2007). Guo and Christopoulos (2011) provides explicit equations for the stiffness and strength:

$$K_{f,i} = \left(\frac{2\pi}{T_1}\right)^2 \frac{\sum_{j=i}^{n} m_j \phi_j}{\Delta\phi_i};\ V_{f,i} = v_f M_e S_a(T_1)\left(\frac{K_{f,i}}{K_{f,1}}\right) g \qquad (1)$$

Where v_f is the normalized strength equal to the base shear capacity of the original frame divided by the elastic shear demand given by the product of the effective seismic mass M_e and the spectral acceleration at T_1. Note that although Eq. (1) is very convenient for getting an approximate strength distribution, in practice, it is rarely reliable unless the building is vertically regular, where the storeys are expected to yield simultaneously. This is because the strength of real frame structures is a function of the plastic mechanism that it develops. Since shear buildings are only simply coupled to the adjacent storeys, once yielding occurs a storey, it cannot transmit additional forces. This is similar to real frames that develop the column-hinging mechanism (storey mechanism), but different from frames that develop the beam-hinging mechanism which still transmits shear forces through flexing of the columns. Often, a more sophisticated procedure such as plastic analysis or a pushover analysis is necessary to obtain a better strength estimate.

For irregular frames, a simple mode shape based index K is used to define the existing vertical irregularity.

$$K = \max\left(\Delta\phi_i \big/ \overline{\Delta\phi_\iota}\right) \tag{2}$$

In Eq. (2), $\overline{\Delta\phi_\iota}$ is the average modal inter-storey drift excluding storey i and any adjacent storeys unless the number of storeys is less than 4, for which the average inter-storey drift, $\overline{\Delta\phi_\iota}$ includes the adjacent storeys. Guo and Christopoulos (2011) proposed a simple rule for describing the inter-storey mode shape for structures with a vertical irregularity at storey i as follows:

1) $\Delta\phi_j^1 = const. (1 \le j \le n; j \ne i, i \pm 1)$
2) $\Delta\phi_i^1 = K\Delta\phi_j^1 \; (j \ne i, i \pm 1)$
3) $\Delta\phi_k^1 = \frac{(K+1)\Delta\phi_j^1}{2} \; (1 \le k = i \pm 1 \le n)$

This mode shape can then be substituted into Eq. 1 to obtain a stiffness distribution that corresponds to index K. Setting K = 1, the target mode shape and stiffness distribution are:

$$\phi_{target}^1 = \frac{H_i}{H_n};\; K_{f,target,i} = \left(\frac{2\pi}{T_{new}}\right)^2 \frac{\sum_{j=i}^n m_j \phi_{target,i}^1}{\Delta\phi_{target,i}^1} \ge K_{f,i} \tag{3}$$

Where h_j is the interstorey height for the j^{th} storey and H_j is the height for the j^{th} floor. The period T_{new} is shorter than T_1 and is obtained by scaling T_1 such that the target storey stiffness is at least equal to the existing stiffness. Assuming the proportionality of stiffness and strength in the target regular building, the target strength distribution is given by:

$$V_{f,target,i} = v_{f,new} M_{e,target} S_a(T_{new}) \left(\frac{K_{f,target,i}}{K_{f,target,1}}\right) g \ge V_{f,i} \tag{4}$$

Where $M_{e,target}$ is the effective seismic mass defined using ϕ_{target}^1 and $v_{f,new}$ is the normalized strength of the new system determined by scaling v_f such that the target strength is at least equal to the existing strength. Eq. 3 and 4 describe the same SDOF to MDOF transformation proposed in Guo and Christopoulos (2012) with the only difference being that the results are scaled up to match the minimum existing stiffness and strength in the original frame. Finally, the added stiffness $K_{a,i}$ and strength $V_{a,i}$ are simply the difference between the target and the existing stiffness and strengths.

From Eq. 3 and 4, unless strategic softening is employed, the target stiffness and strength cannot be lower than the existing stiffness and strength distribution, with the possible exception of the top storey. It is quite clear that step 1 would likely introduce a base shear increase due to the shortened period and added strength, particularly if the irregularity is near the base. Furthermore this increase can be relatively large if the original frame structure is expected to develop large inelasticity since the added damping and strength correction is less likely to reduce the base shear contribution from the frame. For the purpose of examining the 2-step intervention, we will assume that the base shear increase can be accommodated through proper spreading of the added devices across different bays or upgrades in the foundations. However, it is important to note that such upgrades may be either impossible or expensive if foundation strengthening is required. In these cases, a more reasonable solution may be to strategically reduce the strength for storeys away from the irregularity to achieve a more regular base frame.

After step 1, the modified structure is treated as a vertically regular building without further verification. As a result, the P-Spectra procedure for regular structures can be used to assign supplemental damping properties to achieve predetermined performance goals. The P-Spectra is a tool that relates the damped SDOF normalized displacement R_d (damped displacement over undamped elastic displacement) base shear/acceleration R_a (damped base shear over undamped elastic base shear) and residual drift R_s (damped residual displacement over damped displacement) to supplemental damping properties for buildings with known period and normalized strength. The reader is referred to previous works for more details on the P-Spectra (Guo and Christopoulos 2012, Guo and Christopoulos 2013). For the purpose of the current study, it is sufficient to say that the supplemental damping properties (stiffness K_d and activation load V_d for hysteretic dampers, stiffness K_d and viscous damping constant c for viscous-viscoelastic dampers) for step 2 can be obtained using the following:

$$K_{d,i} = \left(\frac{2\pi}{T_{new}\sqrt{\alpha}}\right)^2 \frac{\sum_{j=i}^{n} m_j H_j}{h_i} - K_{f,target,i} \geq 0 \tag{5}$$

$$V_{d,i} = \frac{R_d \Gamma_D S_d(T_{new}) h_i}{H_n}\left(\frac{K_{d,i}}{\mu_d}\right) \geq 0 \tag{6}$$

$$c_i = \frac{2\xi M_D \left(\frac{2\pi}{T_{new}}\right) K_{d,i}}{\sum K_{d,i}(h_i/H_n)^2} \tag{7}$$

where α, μ_d and ξ are the base frame stiffness ratio, hysteretic damper ductility and viscous damping factor that are chosen from the P-Spectra given a set of normalized target responses. The terms Γ_D and M_D are the modal participation factor and effective mass defined using the mode shape $\phi_d = H_i/H_n$ and $S_d(T_{new})$ is the spectral displacement at the new period. Note that Eq. 5-7 are equivalent lateral properties that need to be converted into actual damping device properties appropriately. Note that for fluid viscous devices with very stiff bracing (such that no inclination is intended in the damper hysteresis), $K_{d,i}$ in Eq. 7 is replaced by $K_{f,target,i}$.

3. Validation using shear structures

To get a sense of the general effectiveness of the proposed 2-step procedure, vertically irregular 6-, 9- and 12-storey shear structures defined using Eq. 1-3 with K = 4 is retrofitted using the 2-step procedure. Two locations of irregularities are considered: B-Type irregularity occurs at the base of the structure, and M-Type irregularity occurs at mid-height. Ten ground motions selected from the SAC steel project Los Angeles 10% in 50 year suite (Sommerville et al. 1997) are used for this purpose. The supplemental damping solutions for hysteretic damper and viscous-viscoelastic dampers are selected from the P-Spectra, which is illustrated along with the response spectra for the ground motions in Fig. 2.

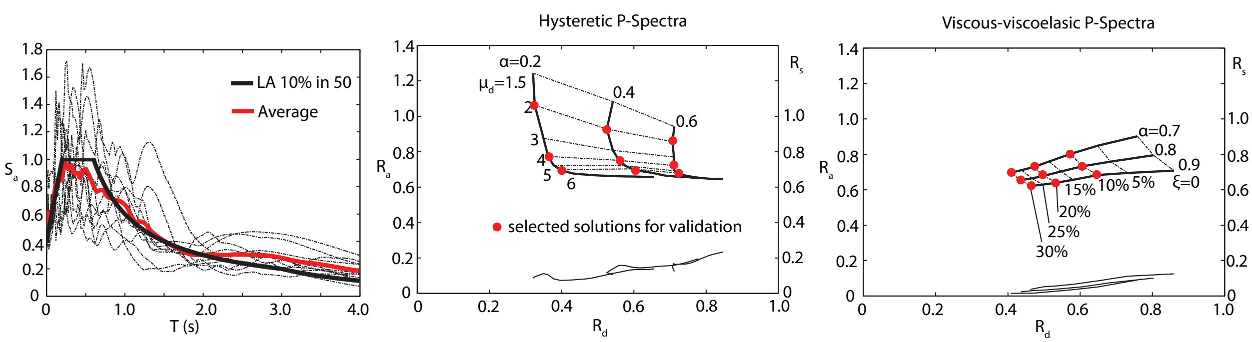

Fig. 2 – Response spectra and P-Spectra target solutions for shear building validation

As illustrated by Fig. 2, the hysteretic damper solutions consist of 9 combinations of 3 different α values and 3 different μ_d values values. Similarly, the viscous-viscoelastic damper solutions consist of 9 combinations of 3 different α values and 3 different ξ values. Furthermore, for buildings of each height, 8 combinations of periods (1.0s, 1.5s, 2.0s, 2.5s and 3.0s depending on height) and normalized strength (20% of elastic base shear, 40% of elastic base shear) are considered. The normalized strength is set relatively low since many irregular structures tends to also have strength deficiency. The average normalized roof displacement R_d, base restoring force R_V and base shear with the viscous contribution R_a are computed from nonlinear time-history analyses and are compared to the P-Spectra targets in Fig. 3. Note that the MDOF responses are normalized to the response of the elastic building with stiffening computed from step 1 of the procedure since this is the new base frame considered for the P-Spectra design in step 2. It can be seen that the procedure results in very good agreement with the P-Spectra targets. The average error in the displacement and total base shear is within 5% although larger errors, up to 25%, in the average restoring force are observed for M-Type irregularities because the procedure strengthens the storeys around the mid-height while slightly weakens the storeys away from the mid-height, included the base of the building. This however, only affects buildings with hysteretic dampers since these systems have a pre-set yield/activation strength whereas systems with viscous-viscoelastic dampers are capable of increasing the base restoring force owing to the supplemental stiffness from the dampers. Other than this, there is no apparent difference between the performance of hysteretic and viscous-viscoelastic dampers used in step 2 of the retrofit procedure.

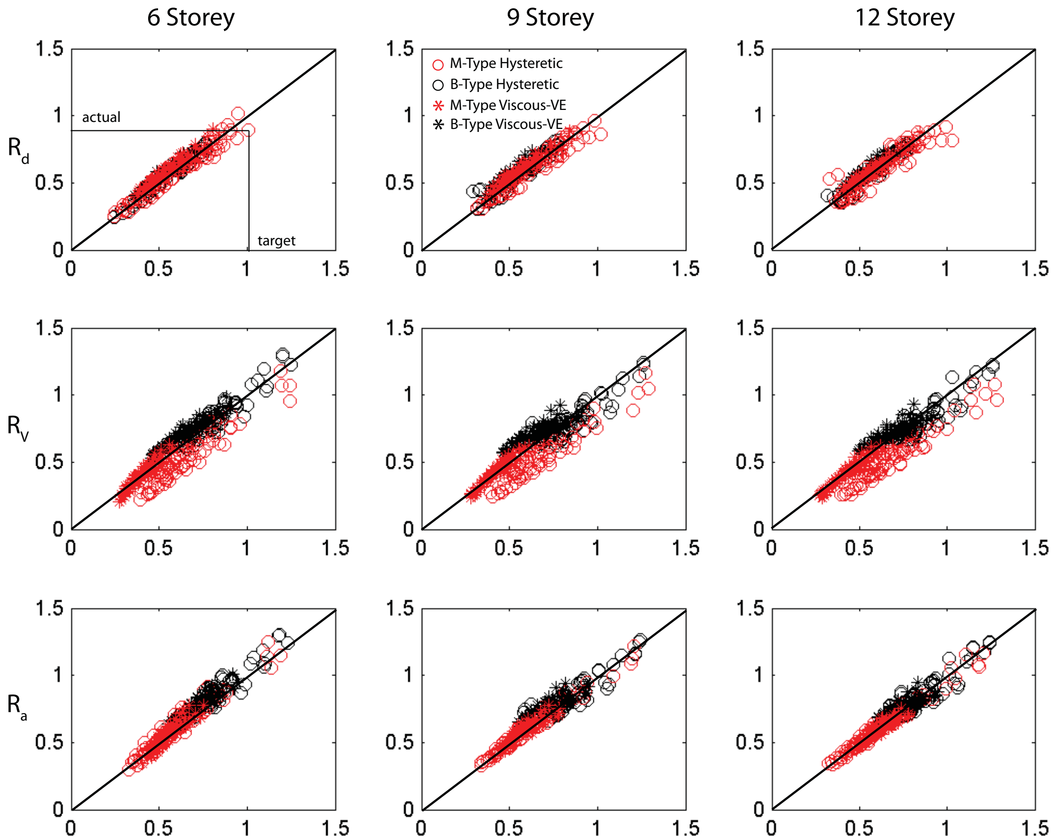

Fig. 3 – Summary of normalized responses of shear buildings compared to P-Spectra targets

Note that the shear building analyses only provide a first validation for the procedure since it only confirms that the global behaviour of the retrofitted structure is as predicted by the P-Spectra. Due to the inherent limitation of shear structures to properly model different frame yielding mechanisms, the storey responses predicted by these simplified models are less reliable. More realistic representations of frame structures should be used for a more rigorous validation and work in this area is being pursued by the authors.

4. Design Example

The 9-storey Type-2 steel frame shown in Fig. 4 is used as an example to illustrate the procedure. As mentioned previously, this building has been studied by Kyriakopoulos (2012) and Guo and Christopoulos (2012) and was found to have a severe stiffness and strength irregularity (weak- and soft-storey) at the first floor. Beam members in this type of structures are only sized for wind load and are expected to yield under the combined action of gravity and wind. This practice was wide spread in the 1970's in Canada, and the same type of building could be built in a seismic zone such as the Canadian pacific west coast. For this reason, the LA suite is used to demonstrate how one may apply the 2-step procedure to retrofit this building using damping devices. Using OpenSees (McKenna et al. 2000), the undamped elastic period of the frame is 2.25s when gravity loads are considered.

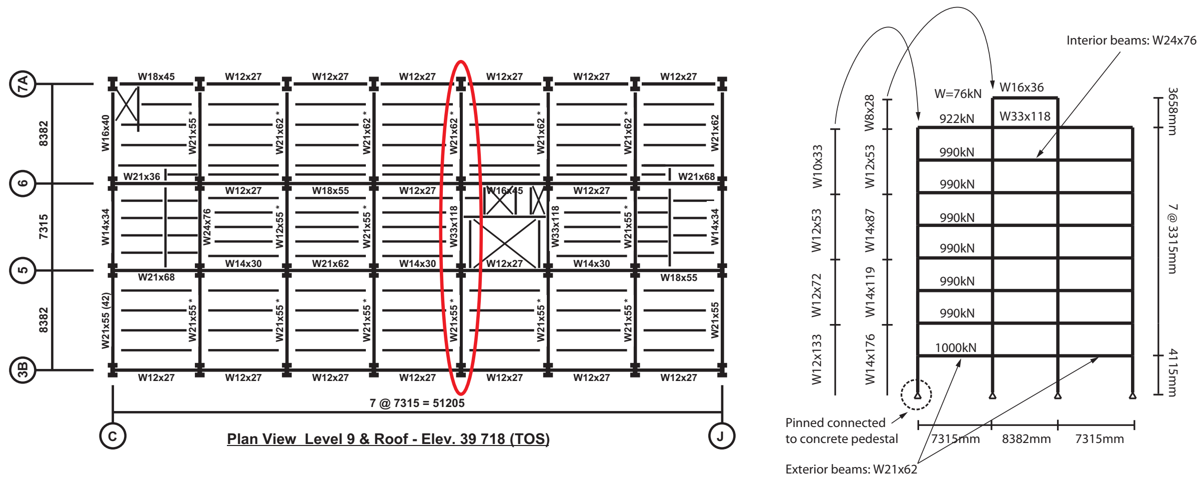

Fig. 4 – 9-storey Type-2 Steel Frame using in design example

The equivalent shear stiffness and strength of this building under the influence of $P-\Delta$ effects have been estimated by Guo (2015) using plastic analysis. Following step 1 of the procedure (Eq. 3 and 4), the added stiffness and strengths are determined and are summarized in Table 1.

Table 1 – Summary of calculations for the stiffness and strength properties for step 1.

storey	$\boldsymbol{\phi_1}$	$\boldsymbol{m}$ (tonne)	$\boldsymbol{K_f}$ (kN/mm)	$\boldsymbol{V_f}$ (kN)	$\boldsymbol{\phi_d = \frac{H_i}{H_n}}$	$\boldsymbol{K_{ft}}$ (kN/mm)	$\boldsymbol{V_{ft}}$ (kN)	$\boldsymbol{K_a}$ (kN/mm)	$\boldsymbol{V_a}$ (kN)
1	0.35	101.8	13.11	511	0.12	87.0	1812	73.9	1300
2	0.50	101.8	29.35	1604	0.24	94.9	1976	65.6	372
3	0.61	101.8	33.44	792	0.35	89.3	1860	55.9	1068
4	0.71	101.8	35.12	917	0.46	81.0	1688	45.9	771
5	0.80	101.8	31.52	602	0.57	70.1	1461	38.6	859
6	0.88	101.8	31.11	706	0.68	56.6	1179	25.5	473
7	0.95	101.8	21.00	371	0.79	40.4	842	19.4	471
8	0.98	93.9	21.61	450	0.90	21.6	450	0.00	0.00
9	1.00	7.8	3.93	130	1.00	2.0	42	0.00	0.00

The added stiffness and strengths can be provided by a metallic yielding system with yield displacement of roughly 15mm to 20mm. To achieve this level of low stiffness, designers would likely have to consider longer support brace elements for metallic dampers. For systems that are expected to yield, the stiffness is less critical than the strength and higher damper stiffness is conservative for design if the strength is matched. For the purpose of this example, dampers are assigned these exact properties. It is worth noting that the original undamped base frame collapses for most of the ground motions in the suite. Hence, the original base shear strength of around 510kN is clearly not sufficient. Following step 1, the corrected frame has a period of 1.23 sec and a normalized strength of 52% (measured with respect to the base shear demand at the new period). It can be seen that at the base, 1812kN is required to make this building regular, while ensuring the strengths at all other levels do not fall under the existing strength. For this particular design, the strength level is governed by the 8th and the second storey. In particular, significant strength is present in the second storey due to the change of effective length of the columns. For this reason, the required strength ends up being much greater than the existing structure. For step 2, a hysteretic damper solution with $\alpha = 0.2$, $\mu_d = 6$ and two viscoelastic solutions $\alpha = 0.7$, $\xi = 30\%$ (VE-stiff) and $\alpha = 0.9$, $\xi = 30\%$ (VE-soft) are chosen. These targets are shown on the P-Spectra in Fig. 2. The calculated supplemental damping properties based on Eq. 5-7 are summarized in Table 2.

Table 2 – Summary of calculations for the added damper properties for step 2.

storey	$\phi_d = \frac{H_i}{H_n}$	Hysteretic		VE-stiff		VE-soft	
		K_d (kN/mm)	V_d (kN)	K_d (kN/mm)	C_d (kNs/mm)	K_d (kN/mm)	C_d (kNs/mm)
1	0.12	347.99	912.29	37.28	10.28	9.67	10.28
2	0.24	379.58	884.59	40.67	11.21	10.54	11.21
3	0.35	357.13	832.28	38.26	10.55	9.92	10.55
4	0.46	324.12	755.34	34.73	9.57	9.00	9.57
5	0.57	280.54	653.79	30.06	8.29	7.79	8.29
6	0.68	226.40	527.61	24.26	6.69	6.29	6.69
7	0.79	161.69	376.82	17.32	4.78	4.49	4.78
8	0.90	86.42	201.41	9.26	2.55	2.40	2.55
9	1.00	6.13	0.00	0.00	0.00	0.00	0.00

Note that the solution VE-soft is actually more naturally and realistically achieved by fluid viscous dampers when it is supported by a finite stiffness. The consequence of the finite support stiffness is that it creates an overall inclined elliptical hysteresis that resembles a viscoelastic damper. In fact, ASCE-41 (ASCE2013) requires fluid viscous dampers to be design as such when the support stiffness is insufficient as determined by testing at the natural frequency of the structure. However, for the sake of demonstrating the procedure, these dampers are modelled as viscoelastic dampers.

Table 3 summarizes the normalized response of these structures as determined by OpenSees. Fig. 5 compares the response of each storey for the step 1 target structure and the undamped elastic base frame. Fig. 6 compares the damped solutions to the elastic frame obtained after step 1. Note that only the elastic response of the original base frame is reported because the inelastic structure collapses for the majority of the records in the suite and would not be useful for drawing comparisons.

Table 3 – Summary of normalized responses of retrofit solutions

	Response	P-Spectra Target	Actual
Hyst.	R_d	0.39	0.39
	R_a	0.67	0.53
	R_s	0.08	0.08
VE-stiff	R_d	0.41	0.46
	R_a	0.69	0.69
	R_s	0.02	0.02
VE-soft	R_d	0.46	0.50
	R_a	0.62	0.71
	R_s	0.04	0.02

In Table 3, the normalized displacements are taken at the roof level, the normalized shear is taken at the foundation level and the normalized residual drift is taken at the roof level. Each of the normalized quantities are computed for individual record and averaged at the end. It can be seen that the mean normalized responses are generally well predicted by the P-Spectra when the target K value is 1.0 (regular distribution) for step 1 of the procedure.

Fig. 5 compares the original frame with the corrected frame after step 1. The severe irregularity at the base of the structure is clearly visible in the interstorey drift plot. Note that even when the building is elastic, the storey 1 drift exceeds 4%. It can be seen that the corrected frame is able to achieve a much more uniform interstorey displacement envelope for both the elastic and inelastic analysis. However, the base shear increase is also very substantial, which highlights the fact that strengthening retrofits, regardless of whether it is done through conventional means or by supplemental dampers, may result in large base shear increase. In this case, the corrected structure has a base shear of roughly 1800kN, which is substantially greater than the original frame strength.

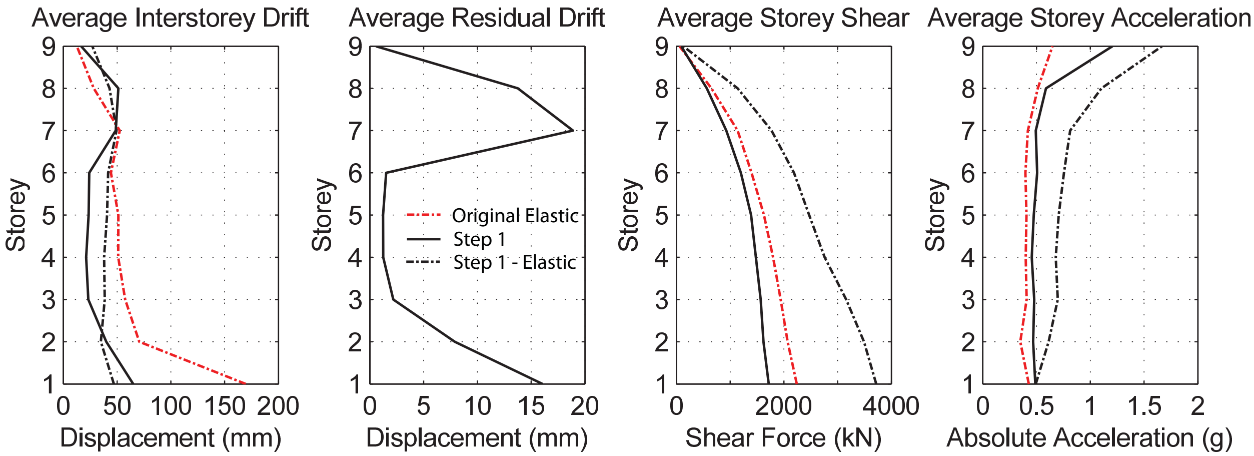

Fig. 5 – Response comparison of original frame (elastic) with corrected frame after step 1

Once the corrected building is defined after step 1, supplemental damping is assigned to further control the displacement response. Note from the P-Spectra in Fig. 2, all three chosen solutions have a minimized displacement response in order to avoid inelastic response in the original structure. Furthermore, these target displacements correspond to the least increase in base shear, or in the case of hysteretic dampers, the base shear is essentially constant across large range of μ_d. The responses of the damped solutions are compared to the elastic response of corrected frame obtained from step 1 of the procedure to illustrate the response reduction indicated by the P-Spectra normalized responses.

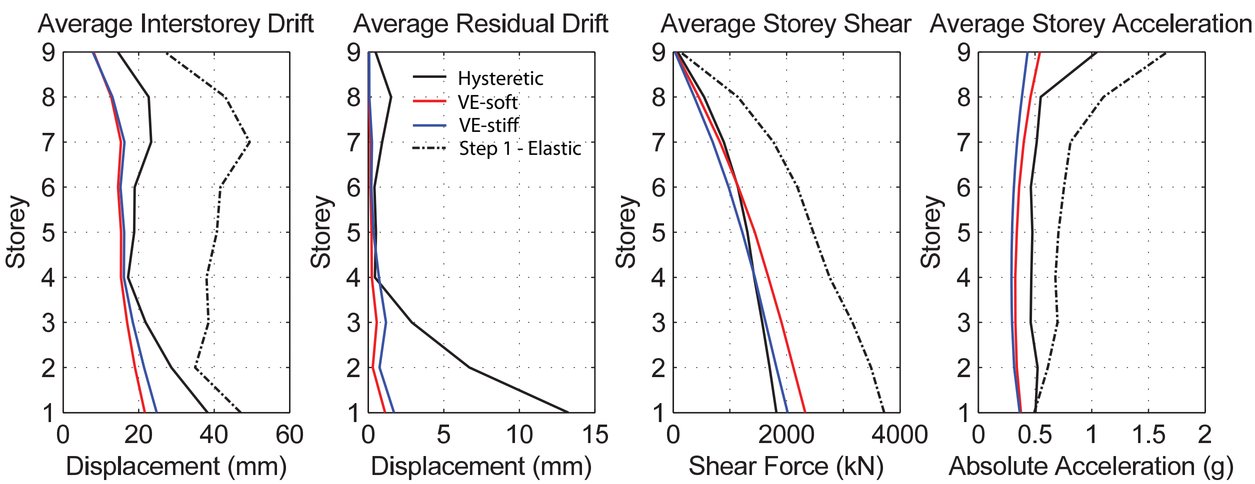

Fig. 6 – Response comparison of corrected base frame with frame with supplemental dampers

Note that the interstorey displacements and shear forces in all three designs are quite close since they were purposely chosen to be similar. It can be seen that generally, the viscoelastic solutions suppresses the storey acceleration better than the hysteretic solution, which is a well-known feature of velocity-dependent dampers. It is also clear that the hysteretic damper solution tends to suffer larger local response amplification for higher intensity motions whereas the viscoelastic damper solutions are able to control local responses better by providing a small restoring force even when the intensity of the ground motion increases. Based on this example, the level of restoring force makes little difference in the performance as long as there is some restoring force. Hence, this indicates that a combination of hysteretic damper in step 1 and viscoelastic dampers in step 2 is able to achieve superior performance in vertically irregular buildings.

5. Conclusion

This paper proposed a 2-step direct design procedure based on the P-Spectra methodology for controlling the seismic response of vertically irregular frame structures and presents preliminary validations using nonlinear time-history analyses of shear structures. It is shown that this approach can effectively achieve the performance targets defined by the P-Spectra by using a combination of supplemental dampers for buildings with irregularities at the base and at the mid-height. Numerical analyses indicate that viscoelastic dampers have the additional benefit of providing a better distribution of storey displacements along the height due to the added stiffness. As a result, better performance for vertical irregular buildings can be achieved by combining supplemental hysteretic dampers, which add strength and stiffness with viscoelastic dampers, which provide post-yield stiffness. An important limitation of this method however, is that both steps of the procedure tend to add strength to the system, which usually result in a retrofitted structure with a significantly higher base shear demand. As a result, this procedure may not be applicable for the retrofit of many older buildings where the existing foundations may not be able to accommodate these forces without serious issues. To circumvent this problem, selective weakening strategies or relaxation of the regularity requirement of step 1 in the propose procedure should be further investigated.

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