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RISK-ORIENTED SEISMIC UPGRADE PLANNING CONSIDERING ADVANCED SEISMIC PROTECTION TECHNOLOGIES – A CASE STUDY

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Abstract

This paper describes a case study for the preliminary seismic upgrade design of a seismically-deficient building using a risk-oriented optimization platform that considers both conventional and supplemental damping systems. The platform combines the P-Spectra and the FEMA P58 methodology for the assessment of building response and post-earthquake impact in order to quantify performance benefits in metrics that are directly relevant to infrastructure owners, and to identify optimal strategies that may otherwise be missed in a traditional code-based retrofit process. The risk-oriented optimization platform relies on P-Spectra, which are rapid performance-optimization tools that display the normalized responses of different solutions combining conventional retrofits (e.g.: strengthening and stiffening) and different commercially available supplemental damping systems. A procedure that converts the P-Spectra responses into FEMA P58-based loss and downtime estimate is then used to create performance maps for different design options. This approach enables engineers to actively and conveniently explore the performance of conventional and supplemental damping solutions at the onset of the design cycle, and make informed decisions that achieve owner's objectives using damping technologies. The case study considers a seismically-deficient concrete building retrofitted using a conventional code-based approach, as well as more advanced viscous and viscoelastic damping solutions identified by the proposed risk-oriented optimization platform. The different retrofit schemes are modelled, and their performance in terms of post-earthquake loss and downtime are evaluated and compared. The study demonstrates that by choosing optimal solutions suggested by the proposed methodology, an increase in the benefit due to reduction in direct losses and downtime compared to a traditional code-based retrofit can be achieved. These results indicate that traditional code-based retrofits using conventional strengthening and stiffening are not necessarily most economical since losses associated with damage and downtime can be high if high-performance retrofit schemes involving advanced technologies are not thoroughly considered at the planning stage.

Keywords: Risk Assessment, Performance-based Retrofit, Supplemental Damping

Introduction

Recent advancements in seismic hazard analysis, structural analysis and damage analysis have transformed performance-based design from a general ideology from its inception a few decades ago, into a more tangible, and rational methodology that engineers are increasingly employing in design offices. Documents such as ASCE-41 [1], FEMA P58 [2] and the REDi guideline [3] provide engineering tools that have been used to design many complex structures around the world where earthquake hazards are eminent. Most of these new guideline documents recognize the positive effect of supplemental damping technologies on building performance and have some quantitative provisions for the use of supplemental dampers in design. However, while these guidelines enable engineers to perform design checks for systems incorporating damping technologies, there is a general lack of guidance to actively steer engineers towards making design decisions that result in the highest performance, measured in metrics relevant to the building owner. In fact, supplemental dampers are often invoked by engineers based on personal experience, or as a patch for making an underlying conventional system work rather than as a conscious and planned design choice arrived by rational consideration of performance. As a result, at the onset of the design process, designers can gravitate towards sub-optimal conventional solutions or miss solutions that may be less costly, and more resilient to damage and loss of function after an earthquake.

To address this problem, a direct performance-based design methodology called Performance Spectra (P-Spectra) Methodology [4] has been proposed to help engineers efficiently estimate the response of a range of structural solutions with or without supplemental dampers in order to select the most suitable options in the preliminary design stage. P-Spectra are plots of the normalized response of equivalent inelastic SDOF systems equipped with supplemental damping devices, which can be used to immediately identify damping solutions that meet predetermined performance targets. Since P-Spectra can be generated with sufficient accuracy for preliminary design purpose using a design spectrum and an estimate of the structural period and strength [5], the procedure is highly cost-efficient. However, although the P-Spectra methodology can be helpful for structural engineers, from the building owner's perspective, the most relevant performance metrics are safety, initial cost, maintenance cost, and the time and financial risk related to earthquakes. When solutions are not compared in terms of these metrics, the owners have little means to justify the use of any advanced damping technologies. In an effort to close this gap, this paper presents a case study on the preliminary retrofit design of a deficient 3-storey office building located in North Vancouver using a risk-oriented retrofit planning method based on augmenting the P-Spectra with a FEMA P58-based loss and repair time assessment framework. Optimal options for retrofit using viscous and viscoelastic dampers considering financial loss and repair time are derived based on performance contours superimposed onto the P-Spectra. This allows designers to directly quantify and explain the reduction of risk to owners, and to consider options involving the use of supplemental damping technologies at the onset of design. It is shown that this augmented P-Spectra methodology can be a useful tool for further advancing and promoting performance-based design, especially with advanced seismic mitigation technologies. This paper first provides a basic introduction to the structure being studied and the seismic hazard at the location of interest. Then, an overview of the P-Spectra is provided, along with a step-by-step procedure for integrating the P-Spectra with the P58 methodology at different seismic hazard intensities. Candidate solutions selected based on the P-Spectra are then designed and modelled in Perform3D, followed by a comparison and discussion of the results.

Building Description

The sample 3-storey office building selected for this study was modified from drawings of another concrete structure prior to its seismic retrofit. The building was originally a concrete frame structure which did not meet the National Building Code of Canada (NBCC) seismic requirements and was at risk of collapse under a design level earthquake with a return period of 2500 years. The building is located in North Vancouver and its schematic is illustrated in Figure 1. The building's original concrete beams and columns were primarily designed to carry wind loads. The diaphragm consists of 200 mm thick precast concrete decks, which can be considered as rigid diaphragm.

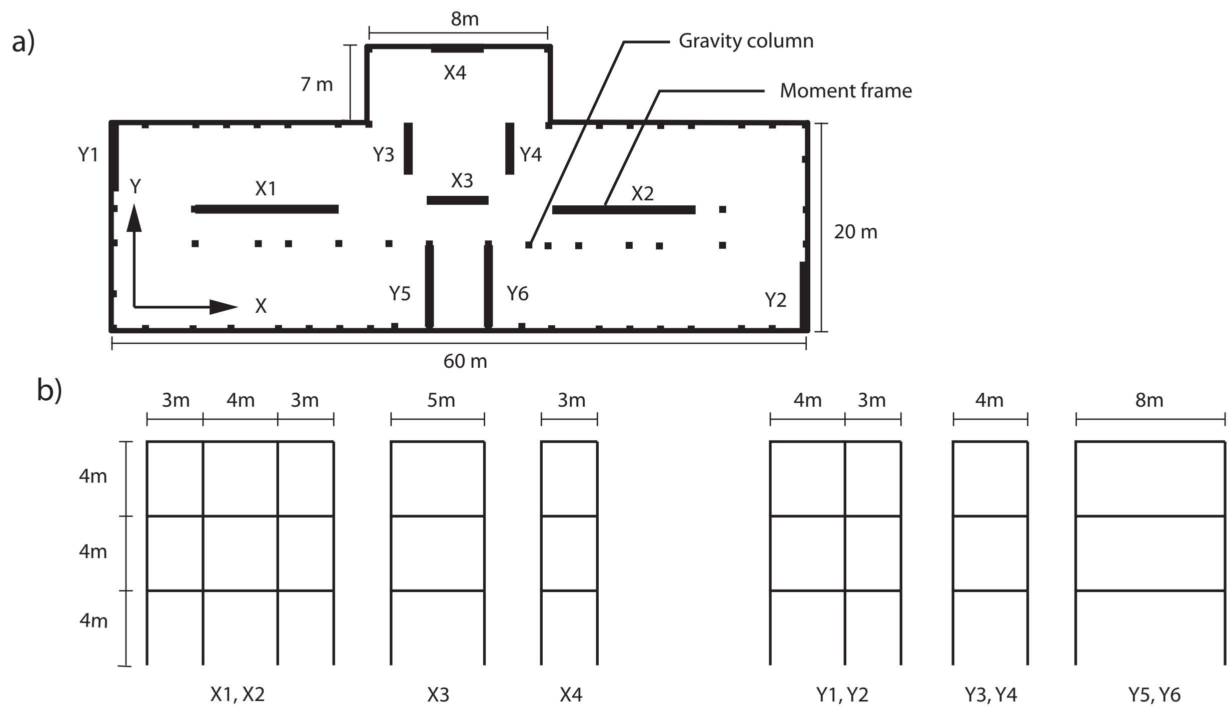

Fig 1 – Case study office building a) typical plan b) elevation of moment frames

As shown in Figure 1, the building has a 20 m by 60 m rectangular plan with the larger dimension in the x-direction. There is a relatively small rectangular block extension on the north side. Since the existing structure does not meet the building code requirements, the foremost requirement of the owner was to bring it to the minimum standard of the National Building Code of Canada 2005 [6]. To do this, the existing lateral force resisting system are upgraded to reinforced concrete (RC) moment frames by strengthening existing concrete beams and columns in the original building as shown in Figure 1a. Although the lateral force resisting elements (RC frames) are asymmetrical in both the x-direction and the y-direction, the center of mass of this building is quite close to the center of rigidity and can be considered to be regular with respect to in-plane torsion. Figure 1b shows also the dimensions of the proposed frames in both orthogonal directions. As mentioned previously, aside from the conventional structural upgrade, a risk-oriented optimization platform based on the P-Spectra will also be used to examine design solutions involving supplemental damping.

Seismic Hazard

The building is located on a soil Category C site (determined in accordance to NBCC2005). In order to facilitate the comparison to a conventional retrofit that was performed for a later study, the seismic hazard as determined by the NBCC2005 was used in this case study. Figure 2 shows the NBCC2005 Uniform Hazard Spectrum (UHS) for the site at a return period of 2500 years, as well as the scaled SRSS response spectra for a suite of 11 pairs of earthquakes selected from the PEER NGA database [7]. The scaling is done in accordance with the ASCE-7 2005 anchored over a period range of 0.2 to 2.0 s. These periods correspond to the periods of the highest mode expected to contribute to the response, and 1.5 times the fundamental period of the most flexible frame considered in this study. These periods are determined from eigenvalue analyses of the structures studied, which are described later. Figure 2 summarizes the earthquakes selected for this study and their individual scaling factor for the 1 in 2500 years hazard. In addition to this, the 1 in 475 years hazard is also considered in this study by applying a uniform factor to the entire suite. Using the 5th generation seismic hazard map of Canada [8], the factor, equal to the ratios of spectral accelerations values at the 1 in 475 years and 1 in 2500 years event is found to be approximately 0.5 at the fundamental periods for the structures considered.

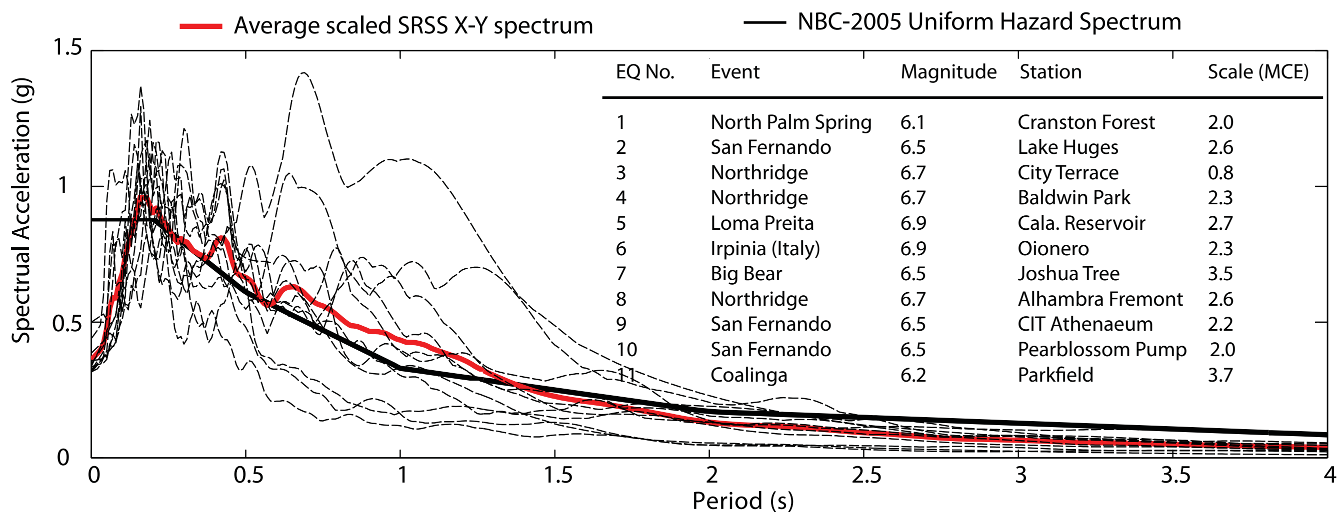

Fig 2 – Design NBCC2005 UHS and scaled suite of 11 pairs of ground motion

Base Frame Design

Since the existing concrete frame does not meet the code requirements for safety, it is first strengthened in selected locations to form a code-compliant seismic force resisting system. Two reinforced concrete moment frame designs occupying the locations X1 to X4 and Y1 to Y6 in Figure 1a are developed and used as base frames (no damper) for the 3-storey structure: the first design uses stiff ductile RC moment frames deliberately sized and detailed to achieve lower peak displacement demands; the second design uses flexible ductile RC moment frames meeting the code minimum base shear. For both designs, the X-direction and Y-direction frame sizes and longitudinal reinforcements (Canadian rebar designation) are the same and these are summarized in Table 1. For brevity, the reinforcement details are not shown.

Table 1 – RC Frame Member Size and Reinforcement

Storey	**Stiff Frame**				**Flexible Frame**			
	Column Size	Beam Size	Column Long. Reinf.	Beam Long. Reinf.	Column Size	Beam Size	Column Long. Reinf.	Beam Long. Reinf.
3	800x700	600x600	5x25M	4x30M	400x400	400x300	5x25M	3x30M
2	800x600	800x600	3x45M	3x45M	500x500	500x400	3x45M	3x35M
1	800x600	800x600	3x45M	3x45M	500x500	500x400	3x45M	3x35M

The first three fundamental periods (translational X, translational Y and torsion) for each frame are obtained using eigenvalue analyses in Perform3D [9]. The first 2 translational periods and mode shapes for each direction, taken at the center of mass at each floor, are summarized in Table 2. It can be seen that the modal properties for the two orthogonal directions are similar for both the stiff and the flexible design. Furthermore, using pushover analyses, the strengths of the stiff and flexible design are verified to be very similar in the two orthogonal directions, and they are taken as 6200kN and 3900kN, respectively. These preliminary results are used later in the P-Spectra analysis.

Table 2 – Modal Properties of RC Frames

Storey	Stiff Frame				Flexible Frame			
	T_{x1} = 0.67	T_{x2} = 0.21	T_{y1} = 0.67	T_{y2} = 0.21	T_{x1} = 1.34	T_{x2} = 0.46	T_{y1} = 1.32	T_{y2} = 0.46
3	1	1	1	1	1	1	1	1
2	0.70	-0.61	0.70	-0.61	0.63	-0.92	0.61	-0.86
1	0.32	-0.83	0.32	-0.83	0.29	-0.78	0.24	-0.75

P-Spectra Loss and Downtime Contour for Performance Optimization

In order to scan different preliminary design options that may result in additional performance benefits, a risk-oriented upgrade planning method using P-Spectra are used to examine and select the optimal supplemental damping options. The optimal solutions found are then designed directly using the P-Spectra methodology [4]. The entire design process requires only modal properties, a strength estimate and a design spectrum. In this study, the designs resulting from the proposed method are subsequently checked using nonlinear time-history analysis. Due to length limitation, only viscous and viscoelastic dampers are examined in this paper to illustrate the procedure. These two damping systems can be characterized by the added stiffness and added damping parameters α and ξ. Other commercially supplemental damping systems can be also considered in exactly the same manner using the corresponding P-Spectra generation methods described in [10] and [11].

P-Spectra are graphs that plot the equivalent nonlinear SDOF system displacement, and residual displacement response against its base shear/acceleration response for a damped structure with a given period T_f and base shear strength v_f, equal to the fraction of elastic first mode base shear demand in the directions of interest. These are fundamental tools in the P-Spectra methodology which can be used to correlate to the actual drift, acceleration, base shear and residual drift response of multi-storey structures with supplemental dampers. The simplified P-Spectra generation method described in [5] is used to generate the viscous and viscoelastic damper P-Spectra for design optimization. Instead of the UHS, the scaled average design spectrum is used, because it better represents the hazard defined by the specific ground motions considered in this study. However, since the P-Spectra response predictions are used to compute separate x-direction and y-direction engineering demand parameters (EDPs) for loss estimation, the SRSS spectrum used for scaling was converted back to the unidirectional spectrum for this purpose. Since the x-direction and y-direction properties are very similar, for simplicity, a factor of 0.707 was applied to the spectrum in Figure 2 for finding the EDPs.

A matrix of P-Spectra for viscous and viscoelastic dampers generated for this purpose are shown in Figure 3 with the periods T_f and normalized strength v_f representing the stiff and the flexible base frame under the 1 in 2500 years and 1 in 475 years events. The normalized strength v_f for each case is computed using:

$$v_f = V_{bf}/S_a M_{eff} \tag{1}$$

where V_{bf} is the base shear strength, M_{eff} is the first mode effective mass, and S_a is the spectral acceleration. Each P-Spectrum in Figure 3 shows a map of feasible supplemental damping solutions with the corresponding mean normalized displacement R_d, mean normalized base shear R_a and mean plus standard deviation normalized residual drift R_s (red curves) as functions of the parameters α and ξ. These response quantities are defined as:

$$R_d = u_{inelastic}/S_d;\ R_a = V_{inelastic}/S_a M_{eff};\ R_d = u_{residual}/u_{inelastic} \tag{2}$$

where S_d is the spectral displacement at the period of interest, M_{eff} is the SDOF system mass, $u_{inelastic}$, $u_{residual}$ and $V_{inelastic}$ are the peak inelastic displacement, residual displacement and base shear of the equivalent SDOF system with period T_f and normalized strength v_f. The coordinates of each point on the P-Spectra give the normalized response of an unique design with different viscous and viscoelastic damping properties.

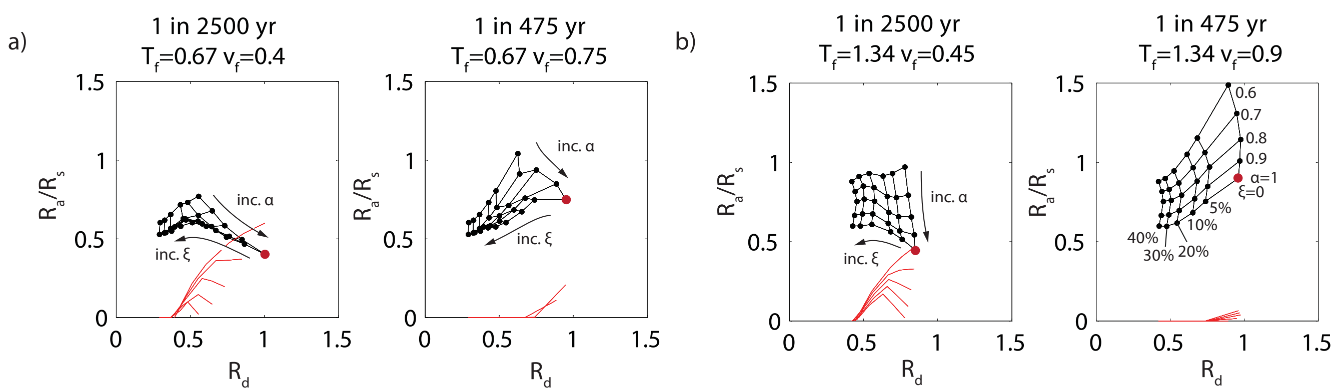

Fig 3 – Viscous-viscoelastic P-Spectra for a) stiff frame and b) flexible frame

For viscous and viscoelastic dampers, the parameter α is defined as the ratio of base frame initial stiffness to the total initial stiffness of the damped system, including the effect of the dampers. Smaller α indicates larger added damper stiffness. The parameter ξ is a constant that is directly proportional to the added viscous damping constant. When the base frame is elastic under the peak displacement demand, ξ is equal to the critical damping. When $\alpha = 1$, dampers only provide viscous damping and do not provide stiffness (i.e. fluid viscous dampers). Hence, when $\alpha = 1$ and $\xi = 0$, there is no added stiffness and damping, which represent the response of the undamped inelastic base frame. This point is colored in red in each of the P-Spectra. As shown in the P-Spectra, increasing ξ always reduces the displacement R_d, and increasing α always reduces the base shear/acceleration R_a but tends to increase the residual drift R_s. The increase in the foundation demand relative to the undamped frames can be found by comparing the values of R_a. It can be seen that under the 1 in 475 years event, the base shear demand is reduced for most supplemental damping solutions. However, the opposite is true for the 1 in 2500 years event where the base frame is expected to develop a full plastic mechanism thus capping off the base shear. Even when $\alpha = 1$, the net base shear can increase when ξ is sufficiently large. Although the foundation demand is not discussed in this paper due to length limitations, it is crucial to carefully consider this for retrofits of existing structures because of the high cost associated with upgrading existing foundations.

In order to obtain risk-based performance metrics, a conversion procedure is proposed to translate the P-Spectra ordinates into meaningful decision variables for owners through the FEMA P58 methodology. To do this, a statistical combination of the inelastic P-Spectra response with the elastic higher mode response is performed to estimate the mean storey response of the buildings with or without dampers. The first mode contribution of response in a MDOF structure is equal to the normalized P-Spectra response multiplied by the corresponding modal participation factor and mode shapes. For every P-Spectra response point, the first mode i^{th} storey displacement and i^{th} floor acceleration responses are computed as follows:

$$D^1_i = R_d \Gamma^1 \Delta\varphi^1_i S_d (T_f) \tag{3}$$

$$A^1_i = R_a \Gamma^1 \varphi^1_i S_a (T_f) \tag{4}$$

In Equations 3 and 4, Γ^1, φ^1_i and $\Delta\varphi^1_i$ are the first mode modal participation factor, i^{th} ordinate of the mode shape, and i^{th} ordinate of the relative mode shape, all computed using the base frame properties obtained from Eigenvalue analyses. The m^{th} ($m > 1$) modal displacements and acceleration responses are similarly evaluated as:

$$D^m_i = R_d^m \Gamma^m \Delta\varphi^m_i S_d (T_m) \tag{5}$$

$$A^m_i = R_a^m \Gamma^m \varphi^m_i S_a (T_m) \tag{6}$$

The response modification factors R_d^m and R_a^m are given by:

$$R_d^m = \exp(-1.35\ \xi_m^{0.5}) \tag{7}$$

$$R_a^m = R_d^m (1+ 4\ \xi_m^2)^{0.5} \tag{8}$$

Equation 7 and 8 are expressions proposed in [5] based on extensive parametric analysis using nonlinear SDOF systems. For systems with fluid viscous and viscoelastic dampers, the higher mode damping ratio ξ_m can be taken as ξ (T_f/T_m), but not more than 1.0. A SRSS combination is used to combine the displacement and acceleration response to obtain the mean interstorey displacement, D_i and acceleration, A_i. Finally, as explained in [4], the mean plus standard deviation residual interstorey displacement is given by:

$$RD_i = R_s D_i \tag{9}$$

In order to use these response predictions for loss analysis, all EDPs are assumed be lognormally distributed with dispersion β_D, β_A and β_{RD}. Since the P-Spectra procedure is an approximation, relatively larger dispersion parameters are applied for the predicted EDPs. In this study, the dispersion parameters β_D, β_A and β_{RD} are interpolated from FEMA P58 table 5-6 [2] to be 0.43, 0.32 and 0.8 for the stiff frame, and 0.43, 0.37 and 0.8 for the flexible frame, respectively. Thus, from the relationship between the median and the mean (or mean plus standard deviation for residual drift) in the lognormal distribution, the median EDPs are obtained as follows:

$$Med(D_i) = D_i \exp(-\beta_D^2/2) \tag{10}$$

$$Med(A_i) = A_i \exp(-\beta_A^2/2) \tag{11}$$

$$Med(RD_i) = RD_i \exp(-\beta_{RD}^2/2)/(1+(\exp(\beta_{RD}^2)-1)^{0.5}) \tag{12}$$

Equations 10 to 12 are used to produce the median demand parameters used for loss estimation in each P-Spectra point. Furthermore, the X-direction and Y-direction demands are assumed to be equal. Finally, the collapse performance of the building is estimated using the USRC [12] recommended translation based on the S-score in FEMA 154 [13]. This procedure assigns an S-score of 3.0 for both the stiff and flexible frames. These scores are converted into a collapse probability based on 10^{-S}, and converted further to account for the difference in the definition of the collapse area in FEMA 154. Since the improvement in collapse performance of viscously damped frames relative to conventional frames is a subject of ongoing research (e.g.: [14-15]), for conservatism, the collapse performance of all damped solutions are assumed to be equal to its corresponding base frame.

Based on these response predictions, the FEMA P58 methodology is used to estimate the seismic losses of the office building and its retrofit options. The FEMA P58 normative quantity estimation tool is used to generate a list of damageable contents for the 3-storey office building. Structural elements such as gravity columns and beam-column joints are also included in the damage model. The median direct loss and the median downtime, taken as the parallel repair time for each P-Spectra point are found using EDPs computed directly from the P-Spectra. The results are superimposed on the P-Spectra generated in Figure 3 as interpolated colored surfaces shown in Figure 4 for the 1 in 475 years and the 1 in 2500 years event, respectively.

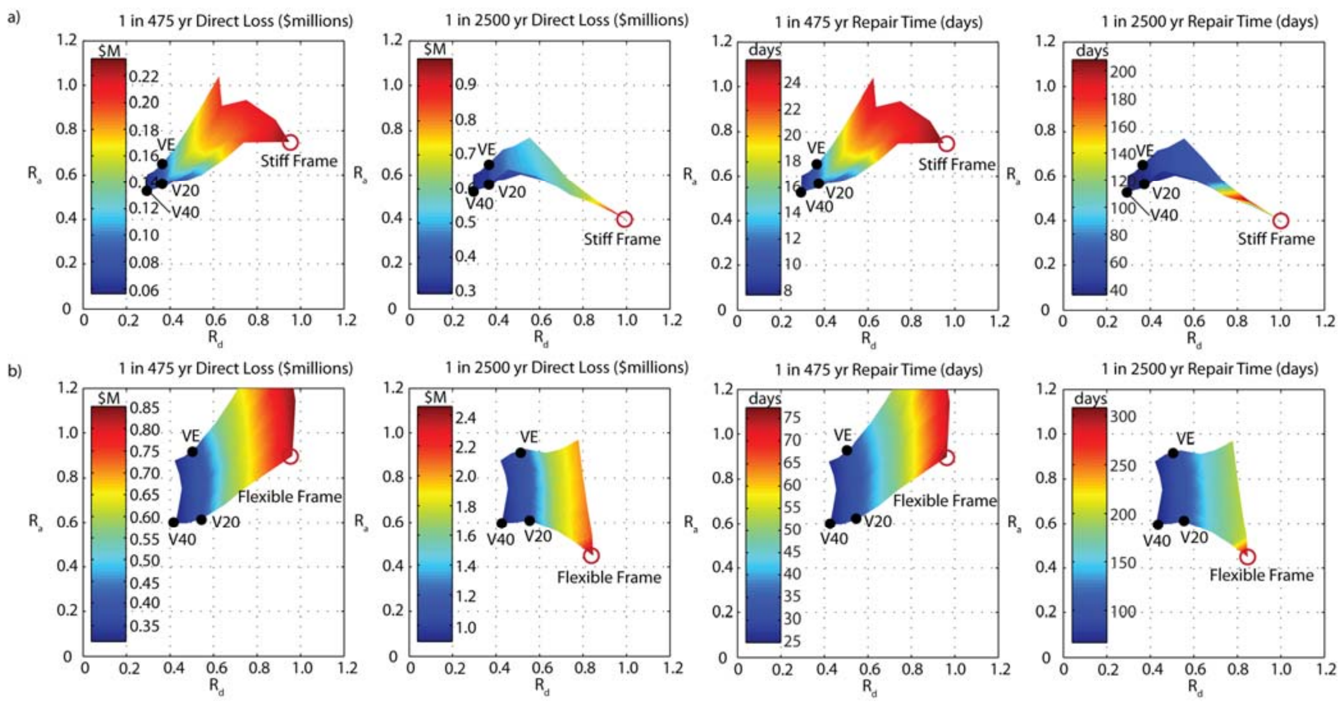

Fig 4 – P-Spectra direct loss and downtime contour for a) stiff frame and b) flexible frame

It can be seen that from a damage point of view, reducing the displacement is the most direct way of influencing the direct loss and down time for both the stiff and flexible design. Examining each solution in more detail reveals that the damage in the wall partition, air handling units and the chiller are the biggest contributors of damage under the 1 in 475 years event. Under the 1 in 2500 years event, the damage to the gravity columns and concrete moment frames also make significant contributions to the loss. Furthermore, it can be immediately recognized that the most heavily damped solutions (smallest displacement) reduce the total direct loss and downtime by about half in the stiff frame, and by about 60% in the flexible frame. Based on the P-Spectra contours, three candidate solutions are selected for each of the frames: fluid viscous dampers with 40% added damping (V40), fluid viscous dampers with 20% added damping (V20) and a viscoelastic damper with $\alpha = 0.6$ (VE), using a commercially available VE material ISD111H (3M Japan). The property of the material under different frequencies of vibration dictates the level of added viscous damping. Based on the material properties at 20C, the corresponding added damping is 20% for the stiff frame, and 25% for the flexible frame. It is expected that the three solutions will suffer similar losses, but the VE solution requires the biggest base shear strength, which in turn may lead to higher cost of floor diaphragm and foundation strengthening. To distinguish the damping solutions for the two different base frames, the prefix "SF" and "FF" are used. For instance, "FFV20" is the flexible frame with 20% supplemental viscous damping.

Using the P-Spectra methodology [4], the required damper size and distribution for the solutions V40, V20 and VE are obtained directly. For this case study, a simple diagonal bracing scheme is used for the dampers in frames X1-X4 and Y1-Y4. Table 3 summarizes the distributed damper net lateral properties (viscous constant C_{lat} and viscoelastic stiffness $K_{ve,lat}$) for these solutions along with the required supporting net lateral brace stiffness K_b for each storey, which are determined based on the rules outlined in [16] for fluid viscous dampers and in [11] for viscoelastic dampers. In order to ignore the effect of finite supporting stiffness in fluid viscous dampers, the required support stiffness in table 3 should be satisfied. However, if smaller support stiffness is desired, a rational modification to the P-Spectra procedure such as that presented in [17] for fluid viscous dampers can be used. Finally, the lateral properties in Table 3 are the sum of the transformed axial properties for all bays in the storey using the corresponding brace angle in the bay. Due to space considerations, individual damper properties are not tabulated.

Table 3 – Summary of damper distribution and designs

Storey	**Stiff Frame**						
	SFV40		SFV20		SFVE		
	C_{lat} (kNs/mm)	K_b (kN/mm)	C_{lat} (kNs/mm)	K_b (kN/mm)	$K_{ve,lat}$ (kN/mm)	$C_{ve,lat}$ (kNs/mm)	K_b (kN/mm)
3	28.0	1400	14.0	700	218	14.3	1750
2	42.1	2110	21.1	1050	329	21.5	2630
1	61.5	3070	30.8	1540	480	31.4	3840
Storey	**Flexible Frame**						
	FFV40		FFV20		FFVE		
	C_{lat} (kNs/mm)	K_b (kN/mm)	C_{lat} (kNs/mm)	K_b (kN/mm)	$K_{ve,lat}$ (kN/mm)	$C_{ve,lat}$ (kNs/mm)	K_b (kN/mm)
3	11.5	575	5.8	290	46	6.7	685
2	22.0	1100	11.0	550	87	12.8	1309
1	33.6	1680	16.8	840	133	19.5	1998

Comparison of Results using Nonlinear Dynamic Time-history Analysis

In order to validate the P-Spectra loss and downtime based methodology that is proposed in this paper, the seismic demands were obtained from nonlinear dynamic analyses carried out using Perform3D. Figure 6 shows sample Perform3D models of the base frame (SS and FF) and of the same base frame equipped with dampers. Note that the original concrete structure is not modelled in this study. However, since it has very limited strength compared to the code requirement, it is assumed that it would collapse and incur much larger losses compared to the cases examined. Although gravity elements that are not part of the main lateral load resisting system can have significant influence on the loss estimation under rare earthquakes by reducing the demolition losses due to excessive residual deformation [18], for the building studied, residual drift was not expected to be a large contributor to losses due to compliance of the base frames with a modern building code. Hence, some gravity elements, for which the reinforcement details are not available, are not considered in the structural and damage model. Beams and columns were modelled using 3-segment frame elements with the fibre sections at the two ends and a linear elastic middle segment. A rigid diaphragm was assumed in the model. Modal damping of 2% in all modes was used with 0.5% Rayleigh damping to ensure stability. The EDPs resulting from the analysis of 11 pairs of ground motions were used to generate the loss estimates. The comparison of the losses and repair time with the predictions made by the P-Spectra are summarized in Figure 6.

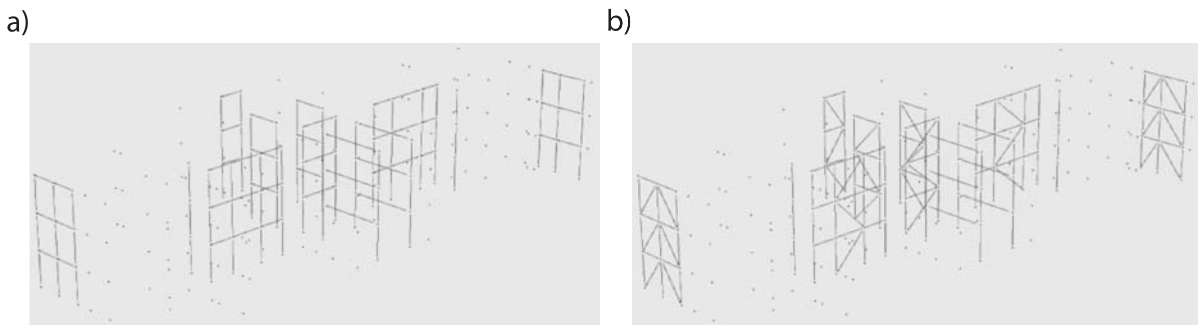

Fig 5 – Perform3D model of 3 storey office a) base frame b) with supplemental dampers

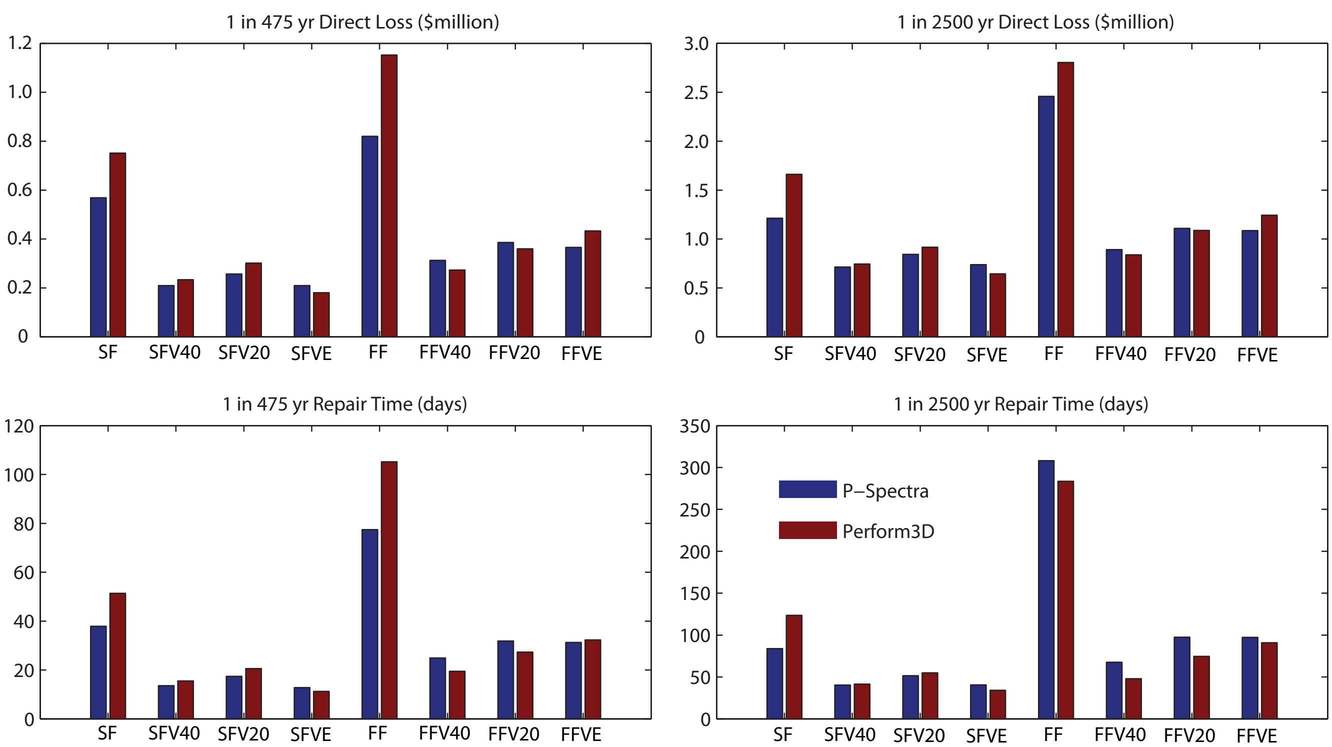

Fig 6 – Comparison of direct losses and downtime computed from P-Spectra and nonlinear time-history

As shown in the figure, the loss and repair time estimates generated by the P-Spectra for each of the solutions compare very well with the nonlinear time-history analyses. For this case, the accuracy seems to be comparable for the 1 in 475 years event and 1 in 2500 years event despite the loss and repair time of the 1 in 475 years event have a much larger relative contribution from acceleration-related damage compared to the 1 in 2500 year event, which was governed by drift-related structural and partition damage. The accuracy of the prediction, which is based on SDOF idealizations of the nonlinear response, is poorer for the undamped structures as there is larger higher mode contribution compared to the damped solutions where the higher modes are effectively suppressed by the added damping. In general, it can be seen that added damping has a greater effect in the flexible frame than in the stiff frame, which is expected as the reduction in displacement was less dramatic in the latter case. Not surprisingly, the best solution in terms of minimizing direct loss and downtime is SFV40. However, aside from the large viscous dampers, which require large supporting braces and larger design forces, this solution is likely to require a more elaborate upgrade to the foundations due to the much larger column axial force demands in the SF case. Hence, depending on the owner's objective, it may be reasonable to select a damped flexible frame, such as FFV20, which is easier to design and more economical to implement due to reduced column force demands, by accepting a 20%-30% increase in loss and downtime. This study demonstrates that the proposed risk-oriented retrofit planning method based on the P-Spectra can be used to guide engineers and owners in making decisions on retrofitting strategies by reviewing all feasible high performance damping solutions at the onset of the design with only a design spectrum and basic dynamic and strength properties of the target structure. Furthermore, the results of the case study indicate that the predicted response based on the proposed procedure correlate well to nonlinear time-history analyses.

Conclusion

This paper presents the risk-oriented preliminary design and performance optimization for the retrofit of a 3-storey office building with supplemental dampers using a P-Spectra-based methodology. A step-by-step P-Spectra-based procedure is proposed for estimating the engineering demand parameters (EDPs) using only a response spectrum and structural properties that are convenient to compute. The estimated EDPs are then used to compute seismic losses and repair costs using the FEMA P58 methodology and the resulting loss estimates are superimposed onto P-Spectra to generate a fast graphical performance optimization tool that explicitly informs owners of the benefits of supplemental damping solution. The proposed procedure is applied to identify and directly generate supplemental damping designs, which are subsequently validated using nonlinear time-history analyses. This study provides preliminary evidence for the validity of the P-Spectra approach for performance-based selection and optimization of design solution at the onset of the design cycle, which can be a useful tool for engineers to actively consider high-performance damping systems. However, several limitations should be recognized. First and foremost, since compliance with modern building codes was a stated goal of this retrofit study, there was very limited risk of imminent collapse in all of the cases that are studied. Hence, whether the P-Spectra EDP demands can be used for buildings subjected to a high risk of collapse remains to be determined. The same can be said about the residual drift demands, which were relatively small and did not play an important role in the determination of seismic loss and repair time for these structures. Finally, this study only presents a single case study with one particular building type and use. In order to further validate the procedure, more work is ongoing, using a variety of building types and subjected to much more severe ground motion inputs to study the accuracy and reliability of the P-Spectra based approach.

Acknowledgements

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References

[1] ASCE (2007). *ASCE Standard No. ASCE/SEI 41-06: Seismic Rehabiliation of Existing Buildings.* American Society of Civil Engineers, Reston, VA.

[2] FEMA (2012). *FEMA P58 Seismic Performance Assessment of Buildings Vol1-2.* Applied Technology Council, Redwood City, CA.

[3] REDi™ (2013). Almufti I and Willford M. *Resilience-based earthquake design (REDi) Rating System.*

[4] Guo JWW and Christopoulos C (2013) Performance-Spectra based method for the seismic design of structures equipped with passive supplemental damping systems. *Earthquake Engineering and Structural Dynamics*, 42(**6**), 935-952.

[5] Guo JWW and Christopoulos C (2013) A procedure for generating performance spectra for structures equipped with passive supplemental dampers. *Earthquake Engineering and Structural Dynamics,* 42(**9**), 1321-1338.

[6] NBCC (2005). *National Building Code of Canada and Commentaries:12th ed.* Ottawa, National Research Council of Canada, ON.

[7] PEER Strong Motion Database: [accessed Mar. 22, 2016: http://peer.berkeley.edu/products/strong_ground_motion_db.html]

[8] 2015 National Building Code of Canada seismic hazard maps. [accessed Mar. 11, 2016: http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/zoning-zonage/NBCC2015maps-en.php]

[9] Perform3D. 3D Performance-based Design Software. Computer and Structures Inc. Berkeley, CA.

[10] Guo JWW (2015) *Performance spectra-based design methodology for buildings equipped with supplemental damper.* Ph.D. Dissertation. University of Toronto.

[11] Guo JWW and Christopoulos C (2016) Response prediction, experimental characterization and P-spectra design of frames with viscoelastic-plastic dampers. *Earthquake Engineering and Structural Dynamics,* DOI: 10.1002/eqe.2732.

[12] USRC (2015). *Implementation Manual: USRC Building Rating System for Earthquake Hazards.*

[13] FEMA (2015). *FEMA P154 Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook 3rd ed.* Applied Technology Council, Redwood City, CA.

[14] Hamidia M, Filiatrault A and Aref A (2014) Seismic Collapse Capacity-based Evaluation and Design of Frame Buildings with Viscous Dampers Using Pushover Analysis. *Journal of Structural Engineering,* 141(**6**), DOI: 10.1061/(ASCE)ST.1943-541X.0001114, 04014153.

[15] Seo CY, Karavasilis T, Ricles J, and Sause R (2014) Seismic performance and probabilistic collapse resistance assessment of steel moment resisting frames with fluid viscous dampers. *Earthquake Engineering and Structural Dynamic,* 43(**14**), 2135-2154.

[16] Lin WH and Chopra AK (2003) Asymmetric one-storey elastic system with non-linear viscous and viscoelastic dampers: Simplified analysis and supplemental damping system design. *Earthquake Engineering and Structural Dynamics,* 32(**4**), 579-596.

[17] Akcelyan S, Lignos D, Hikino T and Nakashima M. (2016) Evalaution of Simplified and State-of-the-Art Analysis Procedures for Steel Frame Buildings Equipped with Supplemental Damping Devices Based on E-Defense Full-Scale Shake Table Tests. *Journal of Structural Engineering,* 142(**6**), DOI: 10.1061/(ASCE)ST.1943-541X.0001474.

[18] Hwang S-H, Elkady A and Lignos D. (2015) Design Decision Support for Steel Frame Buildings through an Earthquake-Induced Loss Assessment. *Proc. ATC & SEI Conference for Improving the Seismic Performance of Existing Buildings and Other Structures.* San Francisco, CA.