Evaluation of Seismic Performances of Flexible Chevron Braced Frames with Fibre-Reinforced Natural Rubber Dampers

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SUMMARY:
This paper presents the development of an innovative configuration of seismic natural rubber dampers for multistorey low- and medium-rise steel braced frames. The dampers are made of fibre-reinforced natural rubber. They are integrated directly in the seismic force resisting system of the structure, connected with typical chevron braces. This control system provides not only additional structural damping to the structure but also a period shift when compared to typical braces, acting in the same way as a base isolation system. A numerical one-storey steel braced building using pseudo linear viscoelastic model for the dampers and a 1/3-scale 3-storey building tested on a shaking table were considered, with and without dampers, to evaluate the seismic performances of the system. Results show the efficiency of the system to strongly reduce the linear seismic forces induced into the structures. Therefore they remain elastic under full-scale seismic intensities, which represents significant economical benefits for the proposed application.

Keywords: seismic isolation; fibre-reinforced natural rubber; damper; chevron braces; multistorey steel frames.

1. INTRODUCTION

Viscoelastic dampers (VED) are usually integrated in a structure as supplemental devices acting in parallel of an already existing and independent seismic force resisting system (SFRS) (Chang et al, 1993; Samali and Kwok, 1995; Housner et al, 1997). The dampers dissipate energy by pure shear deformation and require additional braces in the host structure just acting as supports. This paper investigates the seismic optimization of traditional steel braced frames for low- and medium-rise buildings with a new configuration of natural rubber dampers. The traditional braces are replaced by chevron braces integrating a one-layer fibre-reinforced natural rubber (FRNR) damper horizontally connected in series between the braces and the upper beam of the storey. With this configuration, the dampers are an integral part of the SFRS of the structure. The effect on the dynamic behaviour of the host structure is similar to base isolation: an important period shift is expected due to a reduction of stiffness at each storey combined with a significant increase of the damping ratio of the structure. The main objective of the application is to achieve an important reduction of the seismic forces induced into the structure while limiting drifts and displacements under the current limitations of the National Building Code of Canada (NBCC) (NRCC, 2005) or even reducing them when compared to typical braces. Ragni, Zona and Dall'Asta (2011) recently developed a displacement-based method for the design of a similar system and used it in a numerical study of four-storey and eight-storey buildings.

The proposed system for seismic isolation, based on chevron braces and rubber dampers, is first described. Mechanical properties of the fibre-reinforced natural rubber, obtained from experimental characterization, are then presented. Two case studies of buildings located in Montreal, Canada, are then presented to evaluate the seismic control performances of the system. In the first case study, a 1/3-scale 3-storey steel frame is considered with undamped and damped configurations. The undamped configuration with traditional capacity designed chevron braces is taken as the reference structure and is dynamically compared to the damped structure in which the control system is inserted. This structure was constructed and placed on the Sherbrooke University shake table. The shake table tests and their results, including modal analysis and seismic excitations with reduced intensities, are presented. A numerical evaluation of the seismic behaviour of the damped structure under full earthquake intensities is also presented using a pseudo linear viscoelastic model for the dampers.

In the second case study, a full-scale one-storey steel braced-frame was tested under dynamic loading applied at the top. The corresponding building was also modelled, with and without dampers, and calibrated with the test results. The seismic simulations are discussed and a comparison of the seismic design forces of SFRS members of the damped and capacity designed configurations of the building is presented. Both case studies show the efficiency of the system to strongly reduce linear seismic forces induced into the structure and to keep displacements and drifts under control. The obtained seismic response reduction levels represent significant economical benefits for the application.

2. ISOLATION SYSTEM DESCRIPTION

2.1. Dampers and configuration

Figures 2.1a and 2.1b, respectively, present the configuration of the rubber dampers and their installation in a building. The dampers consist of a unique and thin layer of damping material vulcanized between two thick steel plates. The damper is inserted in a horizontal configuration directly in series between the upper beam of the storey and the top of the chevron braces. The upper plate of the damper is directly bolted to the beam while a WT shape interface section is used to bolt the damper and the braces together. The action lines of each brace intersect at the center of the damping material layer so that the damper is submitted to pure shear deformations during a seismic event.

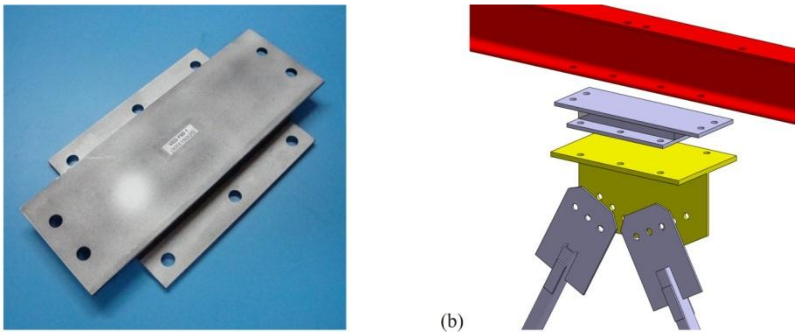

(a) (b)

Figure 2.1. Rubber damper: (a) damper, (b) connection in a frame.

2.2. Damping material

The damping material is a nylon fibre-reinforced natural rubber selected for its stiffness relative to virgin natural rubbers and also because it exhibits a higher damping ratio. FRNR mechanical properties were characterized for cyclic shear deformations. Figure 2.2 presents experimental hysteresis curves obtained for different strains at frequency $f = 4\text{Hz}$. Characterization procedures and results are detailed in Gauron et al (2011). Material shear properties are given in terms of equivalent viscoelastic properties (shear modulus G_{eq} and damping ratio ξ_{eq}) as defined in most of the literature references and design codes (CSA, 2006; FEMA, 2000) for one cycle of the hysteresis curve under harmonic excitation. The main result of the characterization consists in the nonlinear dependence of the equivalent viscoelastic properties of the FRNR with respect to strain as shown in Figure 2.3.

2.3. Advantages of the system

If properly designed, the newly developed isolation system can achieve the following design criteria:

1- The dampers can keep the structure completely elastic by avoiding any buckling of the chevron braces. The cross-section of the braces need not be larger than that of a capacity designed chevron braced frame.

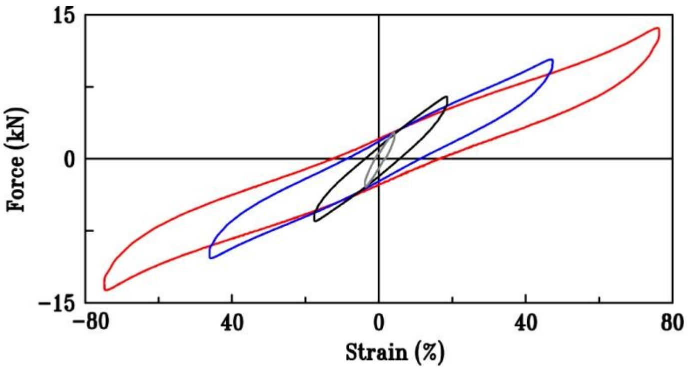


Figure 2.2. Experimental hysteresis curves of FRNR at different strains ($f = 4$Hz).

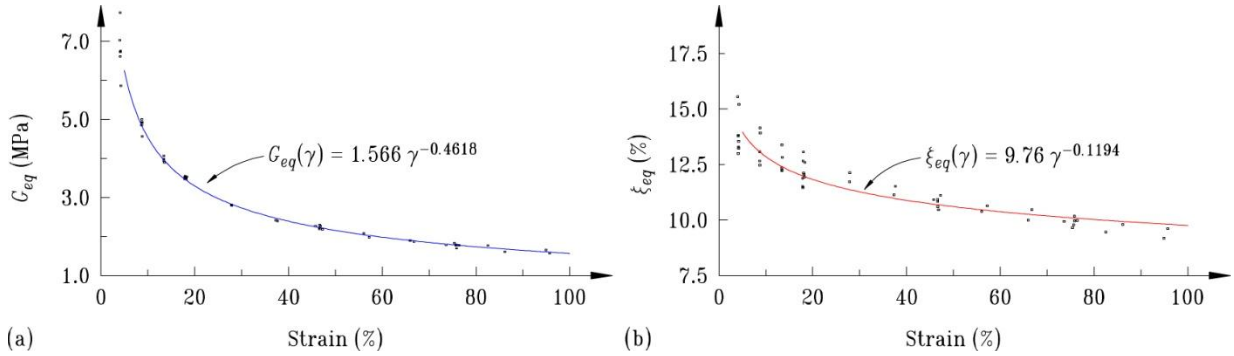


Figure 2.3. Effect of strain on the equivalent viscoelastic properties of the FRNR ($f = 4$Hz):
(a) shear modulus G_{eq}; (b) damping ratio ξ_{eq} .

2- The dampers keep structural displacements and drifts below the limitations of the NBCC (NRCC, 2005), or even reduce displacements and drifts obtained with conventional braced frames, depending on the type of building and the damper design.

The first criterion above translates into a reduction of the elastic seismic forces induced in a conventional braced frame by a factor greater than $R_d = 3$. This corresponds to the ductility related seismic force reduction factor recommended by the NBCC for the design of a typical chevron braced frame with moderate ductility. Both criteria confer many advantages to the system. Safety in the building is assured because structural damage and collapse are prevented. The risk of injuries resulting from non structural damage due to interstorey drifts is also reduced if the dampers are adequately designed. The fact that elastic behaviour is expected with the design earthquake yields important savings with respect to repair or rebuilding costs after a seismic event. Important savings can also be achieved during the design of the SFRS in the case of a new structure, because most of the seismic clauses of design codes based on capacity design philosophy can be relaxed.

3. SEISMIC EVALUATION ON A 1/3-SCALE 3-STOREY BUILDING ON A SHAKE TABLE

3.1. 1/3-scale 3-storey building and test set-up

The dynamic behaviour and the seismic performances of the control system were investigated experimentally on a 1/3-scale 3-storey chevron braced steel frame. This structure was bolted to a uniaxial shake table at the University of Sherbrooke. Figure 3.1a presents an overview of the experimental test set-up. The structure consists of two identical bays in each direction. Its dimensions are 2 m × 2 m in plane and 5.66 m high. In the direction of excitation, the SFRS of the undamped reference structure consists of two conventional chevron braced frames in a “strong column” configuration (Tremblay and Robert, 2000) with tubular sections HSS 38×38×3.2.

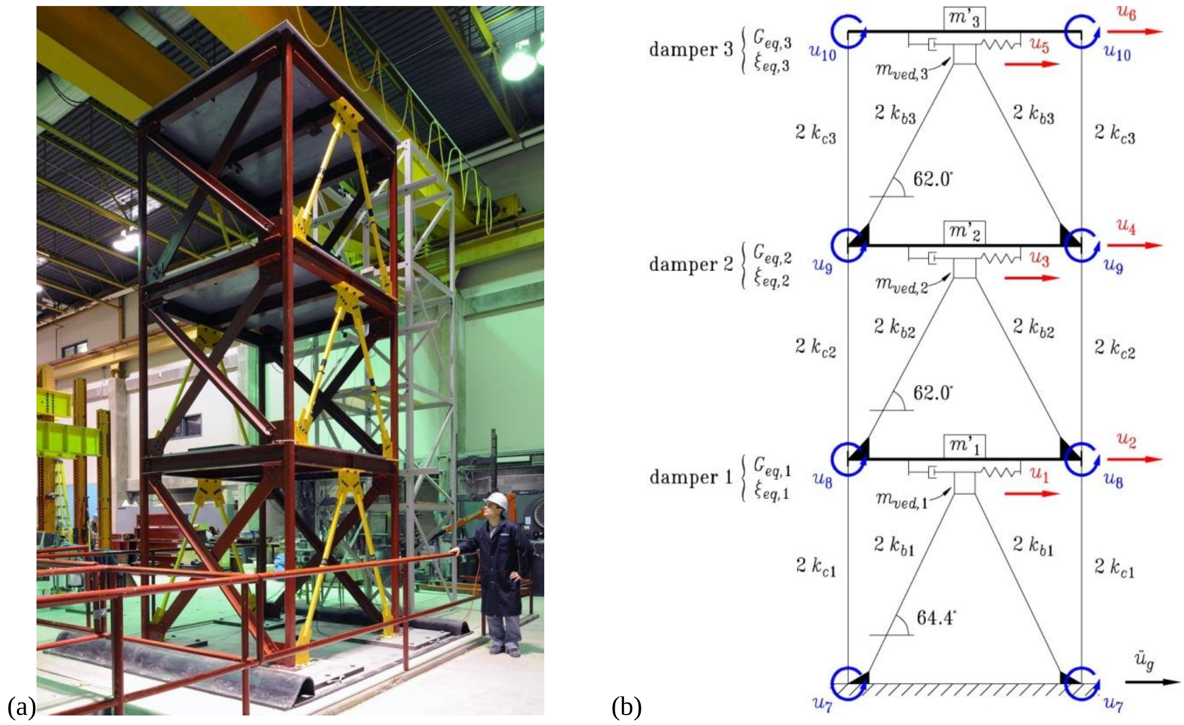


Figure 3.1. 1/3-scale 3-storey building on shaking table:
(a) overview of the test set-up; (b) numerical model of the damped configuration.

Steel plates (1 inch thick) are bolted on each floor to act as rigid floor diaphragms and as a mass source for the structure. Smaller steel plates were also added on each floor to adjust the necessary mass to obtain the desired fundamental period for the undamped structure in the direction of excitation. The first period of the undamped structure was measured at $T_{ref} = 0.112\text{s}$, corresponding to an acceptable value for a 1/3-scale 3-storey steel braced building.

The experimental building was tested with two different configurations. The configuration described above corresponds to a chevron steel braced frame designed in compliance with the seismic provisions of the NBCC 2005 and the CSA S16-01 standard (CSA, 2005) using capacity design. The second configuration includes the new vibration control system, with a horizontal damper inserted at the top of each chevron brace. The dampers were designed using the force-based method developed by Girard et al (2011) and the resulting dimensions of the rubber layers are 300 mm × 150 mm × 20 mm for the first two floors, and 300 mm × 110 mm × 20 mm for the top floor. During the tests, transducers recorded absolute displacements, accelerations and forces in the braces of the structure at each storey, as well as acceleration and displacement of the shake table, and shear deformation of the dampers.

3.2. Numerical study of the seismic performances of the control system

The dynamic behaviour and the seismic performances of the control system in the experimental building were first investigated through a numerical study using *MATLAB*. 2D linear and pseudo linear models were used for the undamped and the damped configurations, respectively (Fig. 3.1b). Modal analysis tests were conducted on the reference structure to evaluate the reference dynamic properties which were used to calibrate the numerical model of the undamped structure. The numerical model of the damped structure is basically the same as that of the reference structure, except that three damper elements are inserted between the chevron elements and the rigid upper beams of the storeys. An equivalent viscoelastic model was used to describe the shear behaviour of the FRNR dampers. This model uses linear equation (3.1) of the perfect viscoelastic model, but it takes into consideration the strong nonlinear dependence of the equivalent viscoelastic properties of the damping material, G_{eq} and ξ_{eq}, related to the maximal shear strain of the dampers as shown in Figure 2.3:

$$F(t) = Ku + \frac{\eta_{eq} K}{\omega} v \tag{3.1}$$

where F and u are the shear force and the shear deformation of the damper element, v is the derivative of u with respect to time, $\eta_{eq} = 2\xi_{eq}$ is the equivalent loss factor of the damping material, ω is the circular frequency of the harmonic excitation of the damper element, and K is the shear stiffness of the damper, such as:

$$K = \frac{G_{eq} A}{h} \tag{3.2}$$

where G_{eq} is the equivalent shear modulus of the damping material, and A and h are the shear area and the thickness of the damper. Equation (3.1) is only valid for a harmonic excitation of the dampers at frequency ω. To use the model for arbitrary loadings such as earthquakes, ω was considered equal to the fundamental frequency of the damped structure which is expected to dominate the response of the structure under seismic excitations. Another simplification, widely used for time integration simulations with elastomeric materials (CSA, 2006; FEMA, 2000), consists in considering equivalent viscoelastic properties for the material corresponding to the maximum strain amplitude experienced during the excitation. The numerical model of the damped structure can be considered as pseudo-linear because values of the equivalent viscoelastic properties of the damping material in equations (3.1) and (3.2) are not constant and depend on the excitation of the structure and its intensity. Calculation of the appropriate values is therefore an iterative process where the stiffness and damping matrices of the structure as well as ω are updated at each iteration step using the maximum strains of the dampers calculated in previous iteration until convergence is reached.

The control performances of the system were evaluated by using the ratio of the maximum responses of the damped structure to that of the reference structure. Variables of interest are: axial force in the braces of storey 1 $F_{ax,1}$, roof acceleration $\ddot{u}_3$, base shear V_0, overturning moment M_0, drift of storey 1 d_1 and roof displacement u_3. The seismic performances were evaluated using eight artificially generated ground motion records developed by Atkinson (2009). These records are compatible with the design spectra of the NBCC 2005 for Montreal (Canada). The time base of the accelerograms was divided by three to respect the scale factor used in this investigation. The average computed control performances for full scale seismic intensities are presented in the first line of Table 3.2. As can be observed, the design criteria of the system are validated: the linear seismic forces induced into the structure are divided by a factor greater than $R_d = 3$ and displacements and drift are also reduced.

3.3. Shake table tests

Modal analysis tests were first carried out on the damped structure, in addition to those already carried out on the reference structure (for model calibration). The table was subjected to a sine sweep at several intensity levels. Only the first flexural mode was investigated to highlight the beneficial seismic effects of the dampers (period shift and increase in damping ratio), and to highlight the nonlinear dynamic behaviour of the damped structure, as a function of excitation intensities. The uniaxial shake table was used to apply a harmonic acceleration at the base of the structure with amplitudes of 0.005g, 0.010g, 0.015g, 0.030g and 0.060g using frequencies close to the fundamental frequency. Figure 3.2 presents the resulting frequency response functions (FRF) of the tests on the damped structure, compared with those obtained on the reference structure. The nonlinear dynamic properties of the damped structure vary with the excitation level. The period shift and the increased damping appear clearly in the results when looking at the progressive shift of the phase change and of the amplitude peak, and also when looking at the progressive and significant reduction of the maximum dynamic amplification factor. In addition it obviously appears that it is impossible to define intrinsic values of the fundamental modal properties for the damped structure due to the

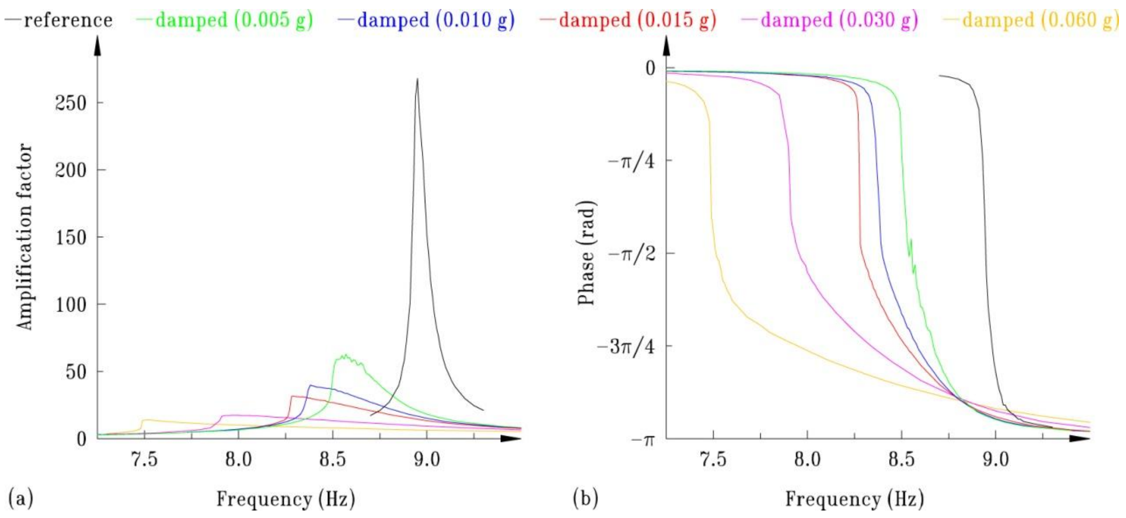


Figure 3.2. FRF of the reference and damped structures: (a) amplification factor, (b) phase difference.

Table 3.1. Results of modal analysis tests on the reference and damped structures

test amplitude	$f_{ved,1}$ **(Hz)**	$T_{ved,1}$ **(s)**	$R_{\max}$
0.005 g	8.54	0.117	63
0.010 g	8.40	0.119	40
0.015 g	8.29	0.121	32
0.030 g	7.95	0.126	17
0.060 g	7.51	0.133	14
reference structure	8.95	0.112	268

nonlinear characteristics of the damping material. Table 3.1 gives values of the first modal frequency of the damped structure $f_{ved,1}$ and of the maximum dynamic amplification factor $R_{\max}$ for each test amplitude on the damped structure compared to the values of the reference structure.

The building on the shake table was also tested using the artificial ground motions. These seismic tests were conducted to determine the experimental values of control performances and to confirm the numerical predictions of Table 3.2. The reference and the damped structures were successively tested with the same artificial ground motion records used in the numerical study. These acceleration records were applied at seven reduced intensities, smaller than the design intensities used in the numerical study. Because the intensity of excitation is a governing parameter for the nonlinear behaviour of the damped structure, the maximum elastic axial forces induced in the braces of the first storey of the reference structure $F_{ax,ref}$ (expressed as a percentage of the elastic design forces C_d) were considered as a relevant parameter to quantify the seismic intensity of an earthquake. In this way, the excitation levels of different earthquakes can be quantitatively compared. The ground motion intensities were selected for each acceleration record in order to obtain seven groups of comparable excitation levels with values of $F_{ax,ref}$ ranging from 5% to 75% of C_d. The reference structure was only tested for the three most reduced intensity levels to verify the accuracy of the undamped numerical model subjected to seismic loadings. Experimental seismic control performances were calculated for each test by comparing the maximum responses measured on the damped structure to the linear numerical responses of the reference structure subjected to the same ground motion. The average values of the seismic performances at each intensity level were calculated to obtain the global performances of the control system. These average values are given in Table 3.2, together with the numerical results for full scale intensities. They are also plotted in Figure 3.3. Nonlinear and continuous improvement of the seismic control performances with an increasing intensity of excitation is a major experimental result confirming the mechanisms of the control system working: an increase of the excitation level is related to a longer fundamental period and a greater structural damping. Both effects are beneficial for the control of seismic forces induced into the system. Both effects have opposed influence, however, on

Table 3.2. Average numerical and experimental control performances of the isolation system

Intensity level	$F_{ax,ref}$ ($\% C_d$)	control performances u_3	d_1	$\ddot{u}_3$	$F_{ax,1}$	M_0	V_0
Numerical	100 %	-28.7 %	-34.2 %	-78.2 %	-77.0 %	-75.0 %	-76.6 %
Level 2	11.2 %	-14.5 %	-21.3 %	-29.8 %	-35.8 %	-26.4 %	-30.4 %
Level 3	13.3 %	-20.6 %	-24.1 %	-34.2 %	-38.5 %	-32.0 %	-33.3 %
Level 4	26.8 %	-31.8 %	-42.4 %	-51.9 %	-56.9 %	-49.2 %	-52.2 %
Level 5	42.7 %	-42.2 %	-49.4 %	-58.1 %	-63.5 %	-55.4 %	-57.9 %
Level 6	57.5 %	-42.0 %	-50.4 %	-65.9 %	-70.2 %	-63.3 %	-66.1 %
Level 7	71.8 %	-45.5 %	-53.1 %	-66.9 %	-72.5 %	-66.1 %	-68.7 %

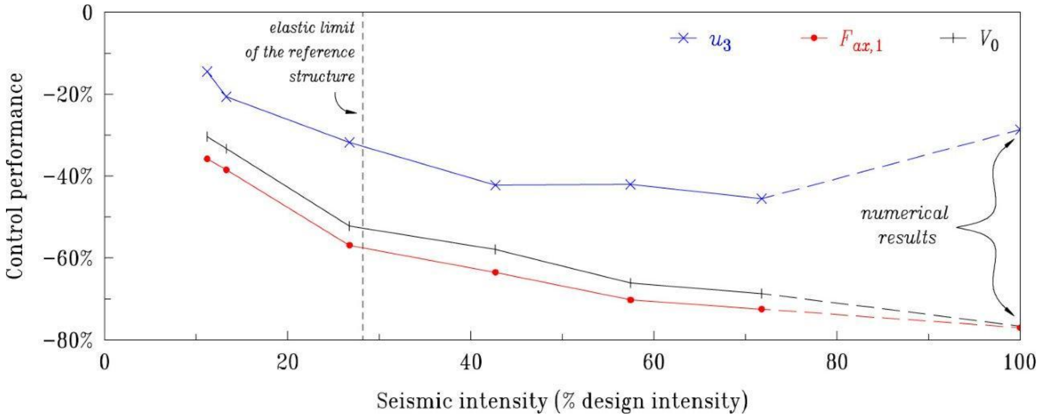


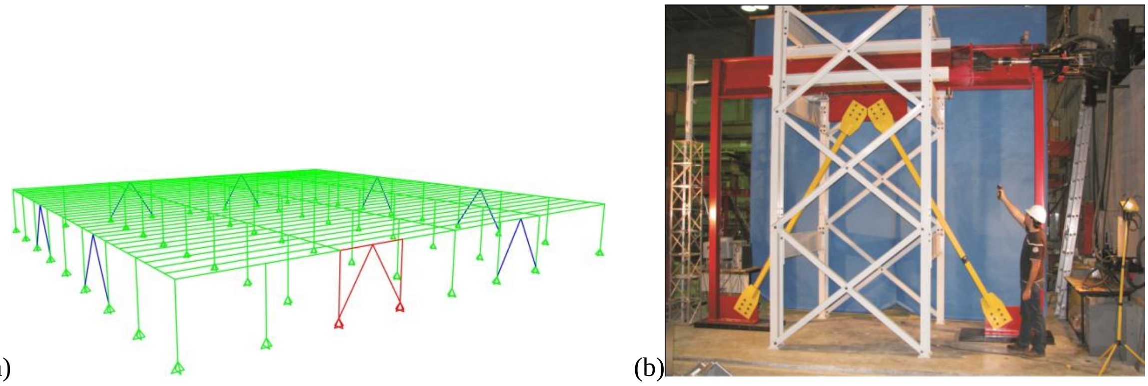
Figure 3.3. Average experimental seismic performances compared to numerical predictions.

displacement control. This is observed in the faster reduction of the control values for these types of variables. The experimental results at level 7 (around 75% of the seismic design intensity) and previous discussion about the evolution of seismic control performances are in good agreement with the numerical predictions for full scale seismic intensities given in Table 3.2. Finally, one of the most important experimental result of this study is that the seismic forces induced in the braces and the induced base shear have been reduced respectively by 72% and 69% at level 7, which is already more than the initial expected reduction factor $R_d = 3$ needed during the design process to keep the damped structure elastic under the full design earthquakes. This has also been achieved with a non negligible reduction of the displacements.

4. SEISMIC EVALUATION ON A FULL-SCALE ONE-STOREY BUILDING

A typical one-storey steel braced building was also considered in a numerical and experimental investigation to study the efficiency and the applicability of the isolation system. The in-plane dimensions of the building are 40 m × 40 m and the storey height is 4 m. Like the previous case study, the building was located in Montreal, Canada, on a soil type C according to the NBCC (NRCC, 2005). A structural damping ratio of 5% was considered according to seismic design provisions of the NBCC for new buildings. The total seismic mass of the building is 370 tons and is concentrated at the roof level. Figure 4.1a presents a 3D-view of the numerical model of the building.

Two configurations of the building were considered and compared for the evaluation of the seismic performances of the isolation system. The building was first designed as a conventional, moderately ductile, chevron braced frame. Dampers were then designed and introduced for the second configuration (damper dimensions: 1200 mm × 80 mm × 25 mm). In both cases, four braced frames were considered in each principal direction of the building. Numerical models of the building were developed using *MATLAB* for each configuration and the pseudo linear model described above was used for the behaviour of the dampers.



(a) (b)

Figure 4.1. Typical one-storey chevron braced steel building located in Montreal (Canada): (a) 3D view of the structural model; (b) Test set-up of the full-scale steel frame.

4.2. Experimental study of a full-scale steel frame under harmonic loading

Before the numerical investigation, full-scale frame representing the damped and undamped configurations of the building were tested under harmonic excitation. The test set-up is shown in Figure 4.1b. Two actuators were used to symmetrically apply loads to the frame. Transducers in the test set-up allowed for the measurement of shear forces applied to the frame, displacements of the structure and shear deformations of the dampers. Tests were conducted with the following objectives:

1- Validate, using the full-scale dimensions of dampers, the material properties, G_{eq} and ξ_{eq}, obtained from the characterization process on small samples.
2- Calibrate numerical models of the undamped and damped braced frames of the building.

The tests on the undamped frame were essentially conducted to obtain the global stiffness of the bracings. Dynamic harmonic excitations were applied to the damped configuration for different displacement amplitudes and frequencies in order to capture the force-displacement hysteretic behaviour of the entire damped steel frame and of the damper. The corrected equivalent viscoelastic properties of the damping material were calculated from the test results. This resulted in the modified equations (4.1) and (4.2). These equations were used during the seismic simulation instead of equations presented in Figure 2.3 to account for the small observed differences:

$$G_{eq}(\gamma) = 1.599\gamma^{-0.5346} \quad \text{(MPa)} \tag{4.1}$$

$$\xi_{eq}(\gamma) = 8.2\gamma^{-0.2525} \quad (\%) \tag{4.2}$$

4.2. Numerical study

Equations (4.1) and (4.2) were integrated into the pseudo linear numerical model of the one-storey building. Figure 4.2 compares the experimental hysteretic behaviour of the damped full-scale frame with computed curves of the model for some of the tests. The figure shows that the numerical model of the frame adequately describes the experimental behaviour for harmonic excitations.

Calibrated models of both the undamped and damped buildings were subjected to the eight ground motion records (used in the first case study) at full-scale intensities. As with the first case study, the seismic performances of the isolation system were evaluated by comparing the seismic responses of the damped building to the linear responses of the undamped structure. The average computed linear responses of both structures in terms of seismic induced roof displacement and base shear are presented in Table 4.1. The damped braced building has an effective natural period 2.5 times longer than that of the undamped building, explaining most of the obtained seismic control performances. Base shear and, consequently, all linear design forces in the elements of SFRS of the damped structure, are reduced on average by 70% compared to the linear seismic induced forces on the

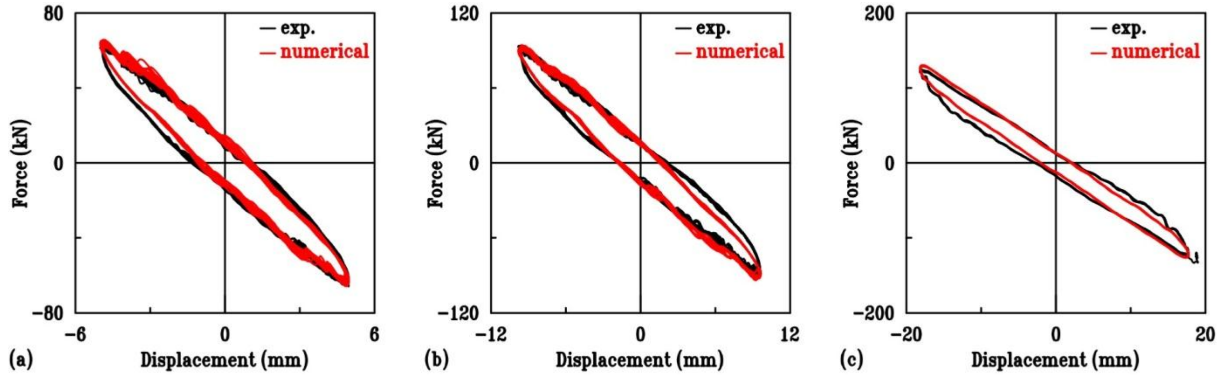


Figure 4.2. Comparison of experimental and numerical hysteresis curves of the full scale damped frame.

undamped building. This is greater than the initially required reduction factor $R_d = 3$ needed to keep the damped structure elastic under full design earthquakes. To illustrate the resulting economical benefits on the construction costs for a new structure designed with the isolation system, Table 4.2 presents the design forces in the elements of the SFRS for the damped configuration of the building compared to design forces obtained by capacity design for the undamped configuration in Table 4.3.

Table 4.1 shows that the average maximum displacements of the damped structure are much higher than the linear displacements of the undamped building in this case study. The displacements remain, however, far lower than the NBCC limitation corresponding to 0.025 times the height of the building ($0.025 \times 4000 = 100\text{mm}$). Results of this study, compared to those of the 1/3-scale 3-storey building presented above, show that the control performances obtained for displacements depend on the actual building, and particularly on the structural damping ratio of the chosen reference structure.

Table 4.1. Average seismic responses of the damped and capacity designed buildings

Structure	Period	Roof displacement	Base shear
capacity designed	0.32 s	12.8 mm	463.3 kN
damped	0.80 s	21.6 mm	141.0 kN
control performance	+ 151 %	+ 69 %	- 70 %

Table 4.2. Design forces in the SFRS of the damped braced frame

Element	Compression	Tension	Shear	Moment
Beam	88.8 kN	88.8 kN	2.0 kN*	2.6 kN.m*
Column	137.0 kN	-	-	-
Bracing connection	177.6 kN	177.6 kN	-	-

* from gravity loads only

Table 4.3. Design forces in the SFRS of the capacity designed braced frame

Element	Compression	Tension	Shear	Moment
Beam	136.9 kN	136.9 kN	163.0 kN	402.4 kN.m
Column	230.6 kN	-	-	8.3 kN.m
Bracing connection	202.5 kN	458.2 kN	-	-

5. CONCLUSION

This paper presented a new configuration of elastomeric dampers for braced frames structures where the dampers are directly integrated in a serial system {chevron braces + damper} that together form the complete SFRS of the structure. Other specificities of the control system are the horizontal and one-layer configuration of the dampers, and the use of a fibre-reinforced natural rubber.

The performances of this new vibration control system were evaluated on two case studies: a 1/3-scale 3-storey building, which was modelled, constructed and tested on shake table, and a full-scale one-storey steel braced building, which was also modelled, and tested under harmonic cyclic loading. Both structures were subjected to eight artificial ground motion records compatible with the Uniform Hazard Spectra of the National Building Code of Canada 2005 for Montreal (Canada). The seismic responses of the damped structures were compared to the linear seismic responses of equivalent chevron braced capacity designed structures. This was carried out to quantify the seismic control performances of the new SFRS. Results of both studies showed that the damped structures remained elastic under full seismic loading. The control system indeed reduces the linear seismic forces induced in the reference structures by a factor greater than the seismic force reduction factor for ductility $R_d = 3$ recommended by the NBCC for moderately ductile concentrically braced frames. The system prevents the host structure from yielding and structural damages. Therefore, the advantages of this new control system are not only related to safety, but also to economical benefits, when considering initial design costs as well as retrofit and repair costs after a seismic events. Results of both studies showed that the system is less effective for displacement control, because the host structures become more flexible, even if they have higher structural damping. The measured displacements, however, were always much lower than the NBCC limits.

Further developments still have to be carried out, in particular concerning the behaviour of the damping material at very low temperatures. The introduction of FRNR dampers in the SFRS could also cause excessive displacements under wind loading. Dampers with small lead cores are therefore investigated in an ongoing research project.

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