INNOVATIVE VISCOUSLY DAMPED ROCKING BRACED STEEL FRAMES

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ABSTRACT :

An innovative braced steel frame system designed to rock at its base under strong seismic motion is proposed. The system includes viscous dampers that are vertically mounted between the foundation and the column bases to dissipate energy and control the structure lateral displacements, while limiting the impact forces induced in the columns. Dynamic cyclic loading tests performed on the individual dampers showed that their load-displacement response could be satisfactory reproduced using a simple nonlinear relationship. Earthquake simulation tests have been performed on a 1:2 reduced scale model of a two-storey viscously damped rocking braced steel frame to assess the overall performance of the system. The rocking response of the test frame could be predicted well using a simple finite element model that includes nonlinear viscous damper and gap elements.

KEYWORDS: Overturning moment, Rocking, Shake table, Viscous damper

1. INTRODUCTION

Reduced seismic design forces are specified in modern building codes (ASCE 2005, NRCC 2005) provided that the structure is designed and detailed to develop a ductile inelastic response under strong earthquakes. This performance can be achieved by applying capacity design principles: designated yielding components of the lateral load resisting systems are carefully sized and detailed to sustain several cycles of inelastic deformation without strength degradation whereas the remaining components of the system are provided with sufficient capacity to resist the maximum forces that will develop upon yielding of the ductile components.

In view of its simplicity and efficiency, steel concentric bracing has been extensively used for lateral resistance in buildings. Lateral loads in braced frames are essentially resisted trough axial loads in the beams, columns, and diagonal braces. The bracing members are designated as yielding elements. Inelastic response develops through yielding of the braces in tension and through plastic rotation in the hinges that form upon buckling of the braces. Beams and columns are designed to resist the maximum axial loads associated with brace yielding in addition to gravity loads in order to maintain their gravity load carrying function upon strong ground motion.

While this design approach can lead to a robust seismic performance and prevent collapse of the structure in case of a severe earthquake, the concept presents several drawbacks that should be considered by building designers and owners. First, capacity design must also be extended throughout the entire lateral load path to ensure that the intended energy dissipation mechanism in the braces can be achieved. For instance, roof and floor diaphragms, as well as the foundations, must be sized to carry the forces associated to the actual lateral resistance of the bracing bents. These amplified design forces can be many times greater than those from the specified design seismic loads. Therefore, the approach can lead to major increase in construction costs compared to past practice. In addition, the design process is more complex and, thereby, more time consuming and costly. A second major issue is the fact that structures so-designed are expected to sustain significant inelastic deformations and exhibit residual lateral deformations after a strong shaking (Fig. 1a). This will necessitate lengthy and costly repairs and, even, total replacement, while creating disruption of the building functions for long periods of time.

Alternative solutions have been recently proposed to mitigate these shortcomings. One approach consists in allowing rocking to develop at the base of the braced frames, as illustrated in Fig. 1b. The base overturning moments can then be considerably reduced, which also reduces the forces induced in the lateral load resisting system. Self-centering capability is provided by the gravity load acting on the laterally displaced structure. Base rotation and lateral deformations can be controlled by means of dedicated energy dissipation devices introduced at the base of the columns, and the structure can be designed to behave elastically, without damage, under the design base earthquake level. The concept has been examined for the seismic rehabilitation of bridges with tall steel truss piers using triangular metallic hysteretic dampers (Dowdell and Hamersley 2000) or buckling restrained braces (Pollino and Bruneau 2007). Shake table testing was performed by Midorikawa et al. (2003) on multi-storey steel structures with base plates detailed to yield in flexure to control the deformations.

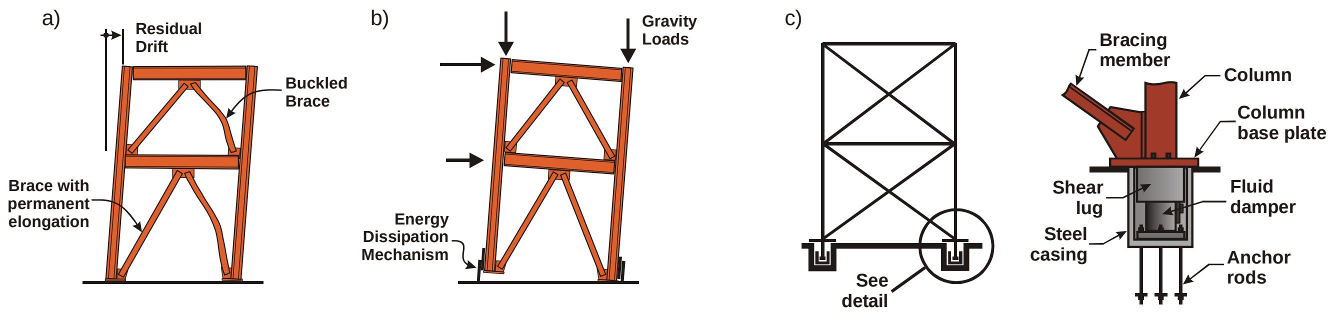


Figure 1 a) Expected seismic damage in concentrically braced steel frames; b) Rocking braced frame; and c) Proposed viscously damped controlled seismic rocking (VDCSR) braced steel frame system.

This paper reports on a study that has been carried out on a Viscously Damped Controlled Seismic Rocking (VDCSR) system developed to enhance the seismic performance while limiting the seismic force demand in building structures. In this system, nonlinear viscous dampers are used to dissipate energy upon column uplift (Fig. 1c). The dampers can be embedded in the foundation, as illustrated in Fig. 1c, or mounted beside the columns, above the foundation. They are designed to control the uplift forces in the columns and, consequently, the overturning moments and lateral loads to be resisted by the structure. They also limit the downward velocity when the columns return to their initial position, which contributes to minimizing impact forces that can be induced upon rocking. The system can be used for seismic retrofit projects as well as for new structures. In the paper, the benefits of using the system are briefly presented for the case of typical low- and medium rise building structures located in Montreal, Canada. A shake table test program was conducted on a two-storey bracing bent model to examine the performance of the VDCSR system and validate the numerical models that are used to predict its seismic response. This test program is presented in the paper. The design of the test model is described. Test results are presented and compared to the results obtained from numerical simulations.

2. SEISMIC PERFORMANCE OF VDCSR BRACED STEEL FRAMES

A parametric study was conducted to evaluate the seismic performance of the VDCSR system for various 2-, 4- and 6-storey building applications. The structures studied had 45 m x 45 m plan dimensions. The influence of the width of the rocking bracing bents (2.81, 5.625 and 9.0 m) as well as their location in the building structures (along the exterior or interior column lines) was examined. For this application, the dampers were assumed to exhibit a nonlinear force-velocity response given by: $F_d = C \cdot v^{\gamma}$, where F_d is the damper force, C is a constant, v is the velocity in the dampers, and γ is an exponent characterizing the nonlinearity of the dampers. In the parametric study, the exponent γ was set equal to 0.25 so that the damper forces rapidly reach a pre-defined maximum load level under high velocity motions, allowing the structure to be properly designed to resist elastically the forces associated to these maximum loads. In design, the damper constant C was adjusted such that the peak inter-storey drifts from nonlinear time history analysis was approximately equal to the Canadian code limit (NRCC 2005). Figure 2 compares peak response parameters for buildings located in Montreal, Canada, when a VDCSR system is used instead of a regular braced steel frame. All structures were located on Site Class C (firm ground) and the analyses were performed under a suite of 12 site specific ground motion records associated to the magnitude-distance scenarios that dominate the 2% in 50 year hazard for the site.

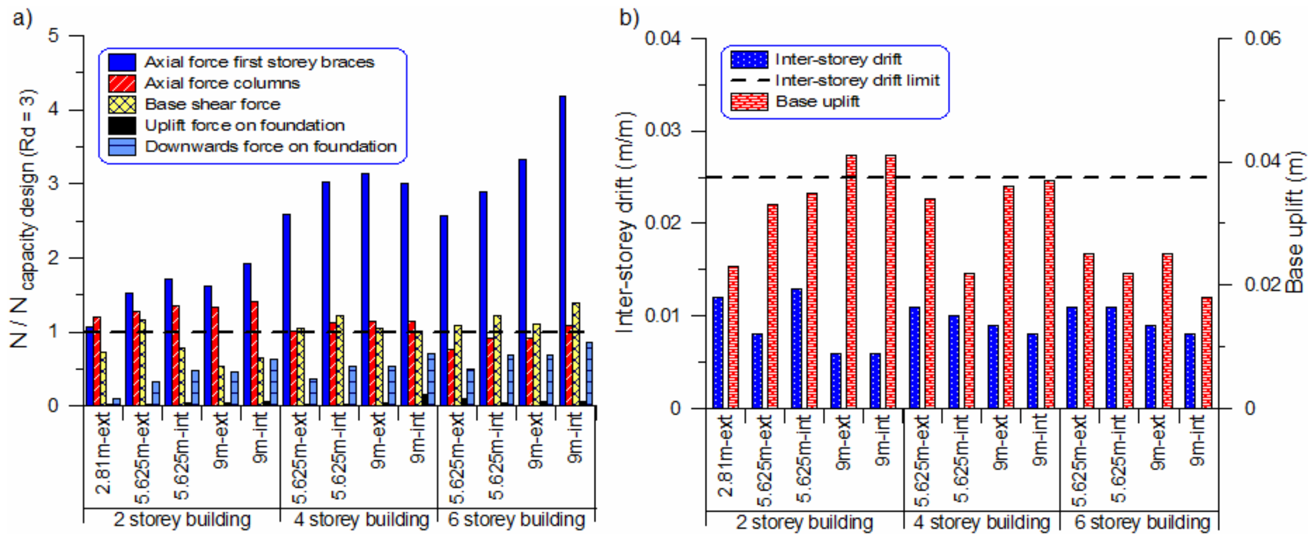


Figure 2 Mean peak response parameters of the VDCSR system applied to 2-, 4- and 6-storey buildings located in Montreal: a) Force demand in the VDCSR system relative to current code capacity design forces; b) Displacement response with the VDCSR system.

In Fig. 2a, the mean values from the time history analyses are normalised with respect to the values obtained from a conventional braced frame designed according to the CSA-S16 seismic design provisions (CSA 2001) assuming a moderately ductile (Type MD) braced steel frame system designed with a ductility-related force modification factor R_d = 3.0. Capacity design principles were applied in the design of these reference braced frames. As shown, the column uplift loads with the VDCSR system are nearly entirely annihilated, resulting in potential cost saving for column anchorage and foundations. Upon rocking, one column of the bracing bent must carry the total gravity loads supported by the bracing bay. In spite of this penalty, peak axial compression loads in the rocking frames remain similar to the forces that must be considered in the design of the corresponding fixed base conventional braced steel frames. For the 2-storey building, the rocking frames generally experience base shears that are comparable to the base shear forces used to size the braces in conventional bracing system. As the building height is increased, the overturning moment response tends to lag behind the horizontal shear force demand and the benefits for rocking gradually diminish. However, braces in conventional braced frames eventually reach their actual buckling and yielding resistances, which results in much larger storey shear forces imposed on the foundation. As shown in Figure 2a, the peak storey shear forces in the rocking systems are comparable to these high capacity design storey shear forces that must be considered in Type MD frames. Figure 2b shows that the uplift of the column bases in all cases studied remained within reasonable limits (less than 40 mm) and that the peak inter-storey drifts are well below the code limit of 2.5% prescribed in Canada (NRCC 2005).

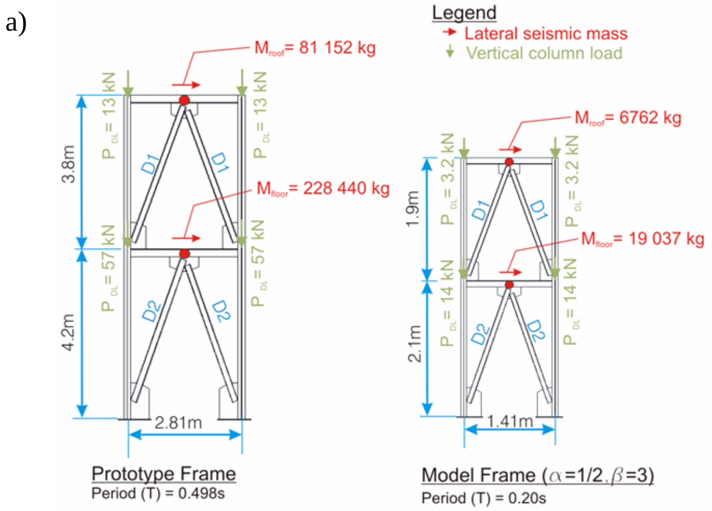
3. SHAKE TABLE TEST PROGRAM

The tests were conducted on the uniaxial earthquake simulator of the Structural Engineering Laboratory at École Polytechnique of Montreal. The multi-cellular shake table is mounted on four frictionless linear hydrostatic bearings. It has 15 ton payload capacity and 3.4 m x 3.4 m plan dimensions. The system features a fully digital three-variable control system with delta pressure stabilization, amplitude phase control, online iteration, and adaptive inverse control capabilities. The test program was conducted on a half-scale model of a two-storey rocking chevron braced steel frame. The test model was designed following strict similitude requirements to fully exploit the capacity of the earthquake simulation facility. The test program also included a characterization of the viscous dampers to validate the assumptions made in the numerical simulations.

3.1 Design and Validation of the Test Model

The prototype braced frame considered for the experimental program is illustrated on the left hand side in Fig 3a. It represents one of the four bracing bents used along the two side walls of a 45 m x 45 m two-storey building structure to resist the seismic loads in one direction. The period of the prototype building is 0.498 s. The braced frame is 2.81 m wide by 8.0 m high and its tributary total seismic mass (roof + floor) is approximately equal to 310 t. The tributary column gravity loads at each floor are given in Fig. 3a.

a)



	Element	Prototype Frame Section	Prototype Frame As (mm^2)	Prototype Frame I_x (mm^4)	Model Frame Section	Model Frame As (mm^2)	Model Frame I_x (mm^4)
Frame Properties	Diagonals D1	HSS 152x152x13	6110	---	---	1528	---
	Diagonals D2	HSS 102x102x13	3790	---	---	948	---
	Columns	W8x28	5310	40.9×10^6	---	1328	2.6×10^6
	Beam roof	W8x10	1900	12.7×10^6	---	475	0.79×10^6
	Beam floor	W8x10	1900	12.7×10^6	---	475	0.79×10^6
Damper Properties	Element	Damping (C) (kN·s/m)	Stiffness (k) (kN/m)		Damping (C) (kN·s/m)	Stiffness (k) (kN/m)	
	Damper	500	3.0×10^6		119	1.5×10^6	

b)

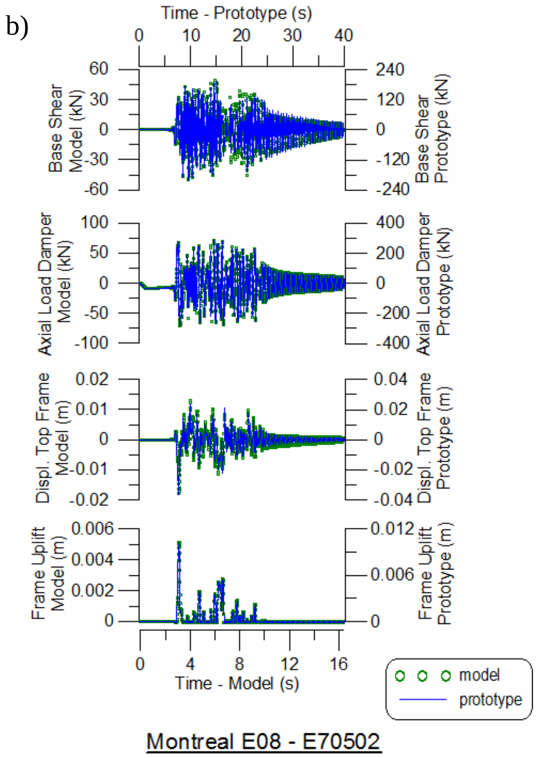


Figure 3 Design and validation of the test model: a) Characteristics of the prototype and model structures; b) Comparison of the response of the prototype and model to an M7.0 at 50 km seismic event for Montreal.

The test model is also shown in the Fig. 3a. It was initially sized according to the artificial mass simulation technique (Moncraz and Krawinkler 1981) using the following dimensionless products: a/g, Δ/L, σ/E, t^2EL/m, ma/EL^2, P/ma, and $C(\Delta/t)^\gamma/ma$, where a is the horizontal acceleration, g is the acceleration due to gravity, Δ is the displacement, L is the length, σ is the stress, E is the steel Young's modulus, t is the time, m is the floor seismic mass, P is the gravity load, and C and γ are the nonlinear properties of the dampers. The scaling factor on length, $\alpha = L_m/L_p$, was set equal to 0.5 (L_m = length in the model; L_p = length in the prototype). The same material (steel) was used for both the prototype and the model, which led to $E_m/E_p = 1.0$ and $\sigma_m/\sigma_p = 1.0$. Because gravity is the same in the test and the actual building, $a_m/a_p = 1.0$ and $m_m/m_p = \alpha^2 = 0.25$. Using this mass ratio, a total seismic mass m_m = 77.4 t was needed to reproduce the 310 t seismic mass of the building.

In order to keep the seismic mass within a level that could be easily and safely handled in the laboratory, the similitude relationships were modified to also include an acceleration ratio $\beta = a_m/a_p = 3.0$. Using this factor, the ratios for mass and time became respectively equal to: $m_m/m_p = \alpha^2/\beta = 1/12$ and $t_m/t_p = (\alpha/\beta)^{0.5} = 0.4082$. The same value was assumed for the exponent γ of the dampers in the model and the prototype structures. This permitted to establish a ratio for the damper constant $C_m/C_p = \alpha^2/(\alpha\beta)^{0.5\gamma}$, which was equal to $C_m/C_p = 0.2376$ when assuming $\gamma = 0.25$. The ratio $(a/g)m$ / $(a/g)p$ cannot be satisfied in the modified similitude procedure as the vertical acceleration due to gravity prevails in both the actual building and the laboratory while the horizontal accelerations in the model are β times the horizontal accelerations in the prototype. A ratio $P_m/P_p = \alpha^2 = 0.25$ was adopted to obtain the correct member forces and stress under gravity loads at the beginning of the test.

Nonlinear time history dynamic analysis of both the prototype and model structures was carried out to validate the assumptions made in the design of the test model. The numerical simulations were performed using the SAP2000 analysis program (CSI 2005) and the simple frame model shown in Fig. 4a. Gap elements acting in parallel with nonlinear dashpot elements are used at the column bases. Horizontal seismic masses were specified at each level. Horizontal and vertical masses corresponding to the gravity loads supported by the frame were specified at the beam-to-column joints to reproduce the vertical inertia loads induced upon rocking. A constant vertical acceleration equal to g was applied during the analysis. Time history response obtained from the prototype and the test model structures are plotted in Fig. 3b. Perfect match was obtained for deformations, column uplift and damper forces. Slight differences were observed for the member forces due to the mismatch between horizontal and vertical accelerations.

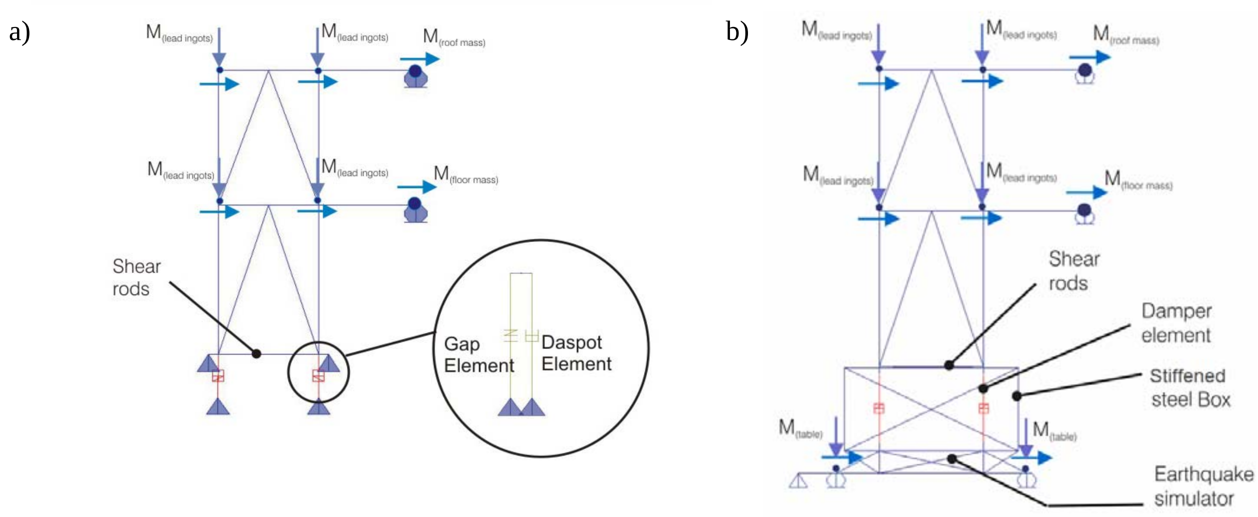


Figure 4 Numerical models used for the analysis of the VDCSR system:
a) Basic model; b) Refined model including the shake table and the stiffened steel box.

3.2 Preliminary Characterization of the Viscous Damper Units

A test program was conducted to assess the mechanical properties of the dampers to be used in the shake table test program. The dampers were supplied by LCL-Bridge Technology Products Inc. The dampers were subjected to various loading protocols including constant displacement rate (velocity) signals, harmonic sinusoidal signals, and damper displacement time histories as obtained from nonlinear dynamic analysis of the rocking braced frames. Figure 5 shows the response of one of the dampers under two different signals. In both cases, excellent correlation could be obtained with the nonlinear model predictions using C and γ values close to the target values from similitude requirements (target C = 119 kN-s/m and target $\gamma = 0.25$).

3.3 Shake Table Test

The shake table test model is illustrated in Fig. 6. The braced frame specimen was mounted on a stiffened steel caisson that simulated the building foundation. The column bases were connected to each other. Horizontal bars were also used between the column bases and the caisson to prevent the horizontal movement of the base plates while allowing column uplift to occur freely. Two viscous dampers were mounted vertically in the base caisson, one under each of the two braced frame columns. At their upper ends, they were assembled to the underside of the column base plates while their lower ends were connected to the earthquake simulator. Concentrated masses built with lead ingots were also attached at all four beam-to-column joints to reproduce the tributary gravity loads supported by the bracing bent at each level.

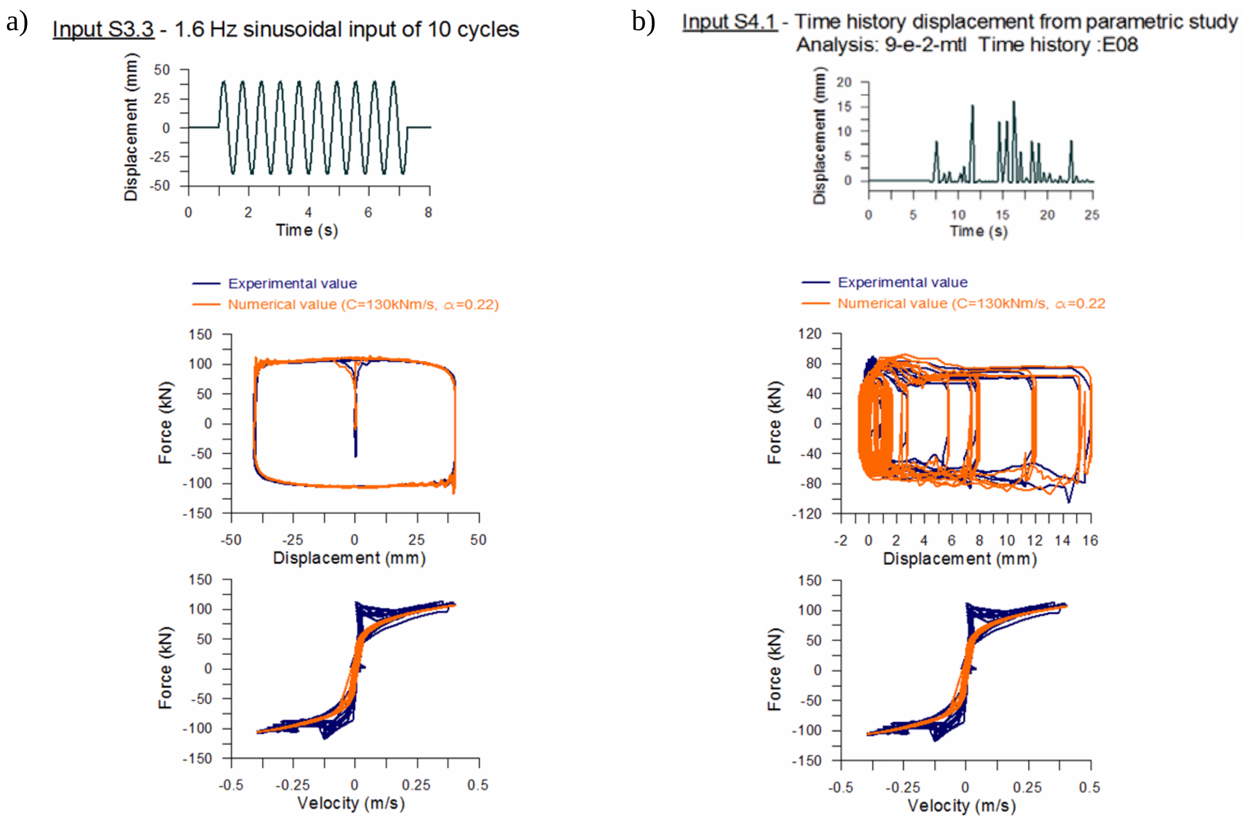


Figure 5 Characterization of the damper units under: a) Harmonic sinusoidal signal;
b) Uplift time histories of the model structure under an **M**7.0 at 50 km seismic event for Montreal.

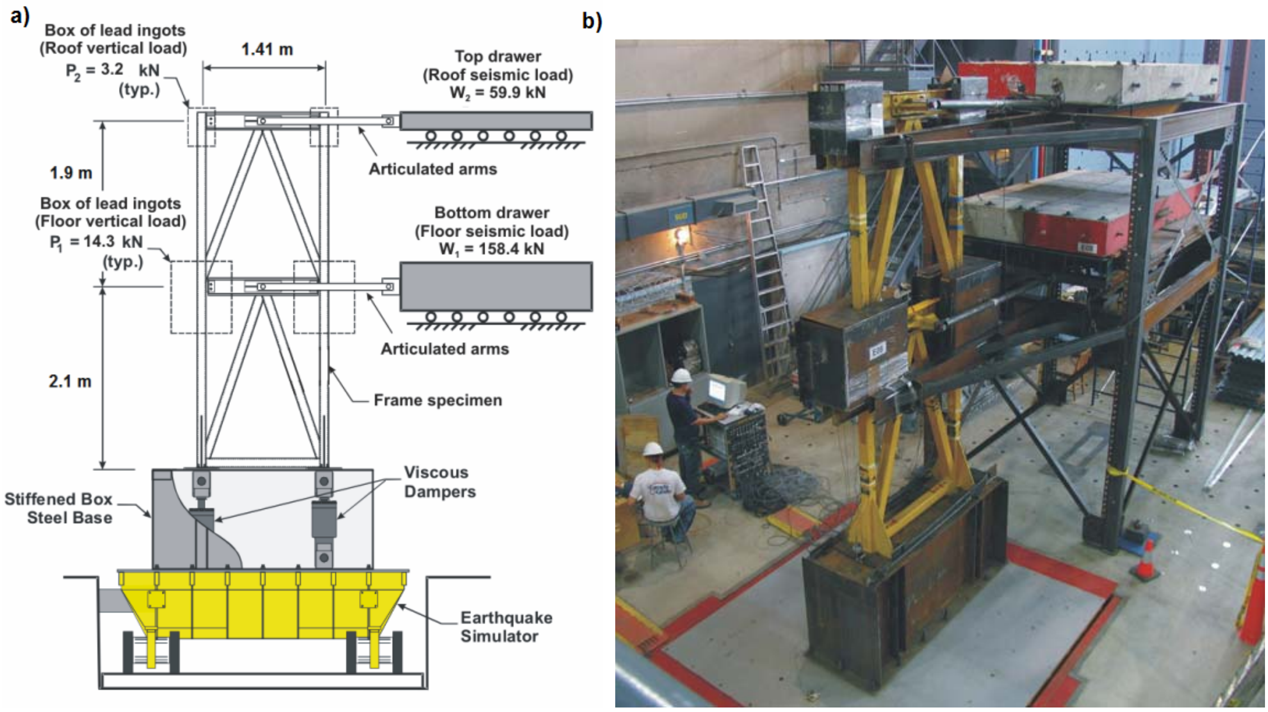


Figure 6. Shake table test setup: a) Schematic of the test model; b) Entire test setup with seismic mass system.

The seismic masses at both levels were made of concrete blocks and heavy steel plates assembled on steel framed chassis that were supported on an isolated two-storey steel framework installed on the laboratory strong floor, beside the shake table. The chassis were mounted on roller bearing units running on smooth stainless steel plates to minimize horizontal frictional forces during the tests. At each level, axially stiff pin-ended struts were used to link the chassis to the test frame.

In the test program, the frame was subjected to earthquake ground motion records representative of three locations exhibiting three different seismic settings: Montreal, QC, representative of eastern North America, Vancouver, BC, located on the west coast of Canada, and Los Angeles, CA, representing the high seismic regions of the western U.S. Three records were considered for each of the sites. In addition, the test model was subjected to harmonic signals with various amplitudes and frequencies. Figure 7a shows the measured response of the test model under a seismic ground motion simulated for eastern Canada and calibrated for a probability of exceedance of 2% in 50 years for the Montreal site. The rocking response of the frame can be clearly observed from the test results. The relative displacement (*u*) and the rotation of the two levels are in phase, indicating that the motion of the frame is dominated by rocking. For this particular ground motion record, uplift displacements up to 19 mm were measured at the column base, resulting in peak inter-storey drifts of approximately 1.5%. The VDCSR system behaved as intended in design, without structural damage, under the entire series of harmonic and seismic signals that were imposed.

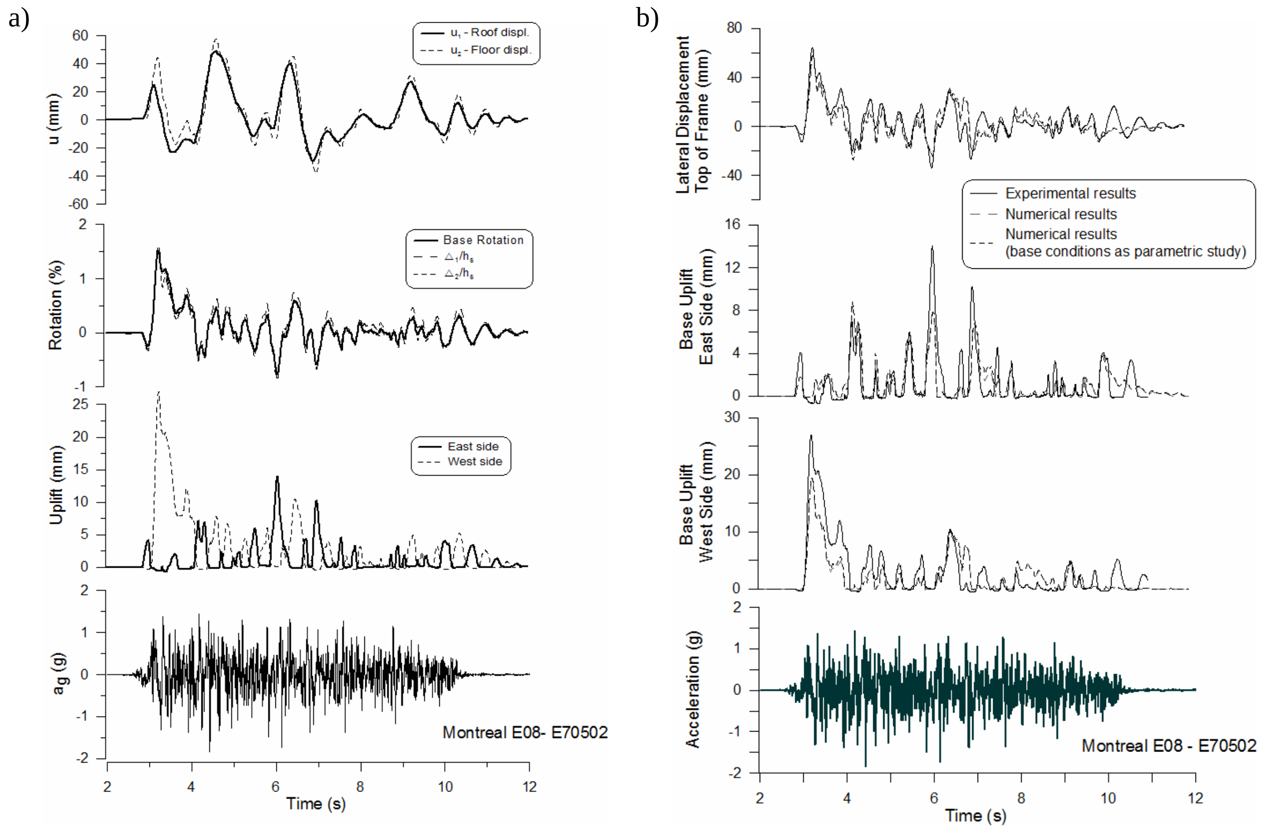


Figure 7 Shake table test under an **M**7.0 at 50 km seismic event for the Montreal site:
a) Measured response; b) Comparison between test results and numerical predictions.

The main objective of this test program was to validate numerical models used to predict the response of actual building structures equipped with the VDCSR system. In Fig. 7b, the lateral deformation and column uplift predictions from two numerical models are compared to the measured response for the same **M**7.0 at 50 km ground motion. The first model is the one presented in Fig. 4a, i.e. the model that was used for the parametric study of the building seismic performance as well as for the design of the scaled test specimen. The second model is illustrated in Fig. 4b. Additional members were introduced to more closely represent the base conditions that prevailed in the test setup, including the stiffened steel caisson and the shake table. In Fig. 7b, the results from both numerical models are nearly identical. This is attributed to the fact that the frame deformations were essentially governed by the rocking response and that rocking did not seem to be sensitive to slight variations in support conditions. The responses from both analytical models are also in very good agreement with the test data: occurrences of uplift at the base of both columns could be very well predicted as well as the large amplitude structural lateral deformations. Similar correlation was obtained for the other ground motions. This suggests that simple analytical models currently available in design environment can be used with confidence to predict the deformation demand on rocking braced frames equipped with nonlinear viscous dampers. At the time of writing, validation of member force and acceleration predictions was still ongoing and conclusions on this aspect are yet to come.

4. CONCLUSION

The paper described an innovative viscously damped seismic controlled rocking braced steel frame system that is proposed as a cost-effective solution for enhanced seismic response of new and existing structures to severe earthquake ground motions. The performance of the system was examined through analytical and experimental studies. The numerical simulations for building samples located at the Montreal site indicated that the system can lead to significant reductions in column uplift loads compared to conventional braced steel frames. Reduction in horizontal shear forces is also expected in low-rise building applications. All structures studied could sustain the design ground motion demand without structural damage and the peak lateral displacements remained within prescribed code limits. The properties of the viscous dampers were verified though dynamic testing. Shake table tests conducted on a half-scale model of a two-storey rocking braced frame confirmed the adequacy of the numerical models used to predict the response of the VDCSR system.

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