Segmental elastic spine system for enhancing the seismic response of tall EBFs in eastern Canada

Liang Chen[1], Robert Tremblay[2], Lucia Tirca[3]

[1] Ph.D., Research Assistant, [3]Associate Professor, Department of Building, Civil and Environmental Engineering, Concordia University, Montreal, QC, Canada.
[2] Professor, Department of Civil, Geological and Mining Engineering, Polytechnique Montreal, Montreal, QC, Canada.

ABSTRACT

Conventional EBFs are prone to soft-storey mechanism due to their lack of vertical continuity between storeys. Studies have shown that this behaviour can be more pronounced when the resistance of the ductile links does not uniformly match the design seismic forces over the height of the frame, as can be the case when wind loading requirements control link design over a portion of the building height. This situation is expected for tall buildings located in regions with low or moderate seismicity encountered in eastern Canada. The braced frame with segmental elastic spine (SESBF) system is a new EBF configuration in which segmented vertical elastic trusses are introduced to provide flexural continuity along the frame height and prevent concentration of inelastic deformations. The article illustrates the improvement of seismic response that can be achieved with the SESBF configurations for a 24-storey building located in Montreal, QC, exhibiting non-uniform link seismic overstrength. Two SESBF configurations are studied. Both the conventional EBF and SESBF structures were designed in accordance with the NBCC 2015 provisions and wind-induced forces governed the link design in the first 20 storeys. The seismic response of the frames is examined through nonlinear time-history analysis. The EBF showed unacceptable seismic response with excessive storey drifts and structural collapse in the upper levels. The two structures with the SESBF system exhibited a stable seismic response with storey drifts and link plastic rotations well within code limits.

Keywords: Eccentrically braced frame, Elastic spine, Eastern Canada, Seismic, Wind.

INTRODUCTION

The eccentrically braced frame (EBF) system is well-known for its large lateral stiffness and high ductility. During an earthquake event, the input seismic energy is dissipated through yielding in shear and/or bending of ductile link beam segments. For this system, seismic design provisions require that inelastic deformations be limited to the ductile links, which are designed and detailed to sustain the expected inelastic demand (Figure 1). All other structural members must be designed with sufficient resistance to remain essentially elastic during earthquakes.

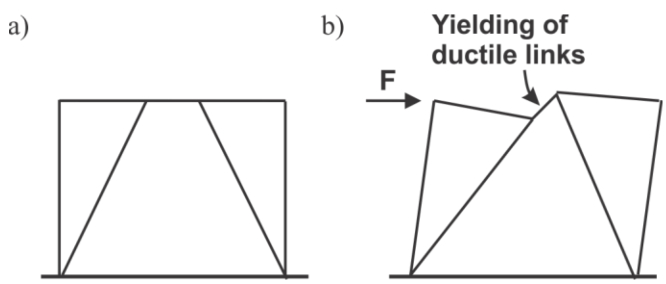

Figure 1 Eccentrically braced frame (EBF) system: a) Typical configuration; b) Intended yielding mechanism.

Despite their appealing characteristics, concentration of inelastic deformations within a single storey of multi-storey EBFs was observed during past experimental and numerical studies [1-7]. This soft-storey mechanism is illustrated in Figure 2a. Popov et al. [8] has pointed out that this concentration of inelastic deformation is mainly due to ill-proportioned link sections within the structure. He suggested that the link sections be proportioned such that their resistance closely matches the seismic force demand at every storey along the frame height to achieve a desirable seismic performance and uniform inelastic deformation demand. Incorrect proportioning of the links can result in a damage concentration in particular storey levels.

To overcome this drawback of conventional EBFs, researchers have proposed to vertically tie the links in adjacent storeys such that link yielding can develop simultaneously in several storeys rather than concentrating in one or a few levels [1, 9-13]. This system is very effective for achieving uniform storey drift response but this is at the expense of large forces developing in the tie and other members. Chen et al. [14, 15] has recently proposed a modified approach in which the vertical ties are used only in segments of the structure height. The resulting segmental elastic spines are found to be sufficient to prevent concentration of inelastic demand while reducing the force demands on the structure components. This new structural system, referred to as SESBF, is illustrated in Figure 2b. Within each segment, the vertical ties connecting adjacent storeys form an elastic truss with the connected beams, braces and columns. These elastic trusses from spines that can effectively mitigate the concentration of inelastic deformations.

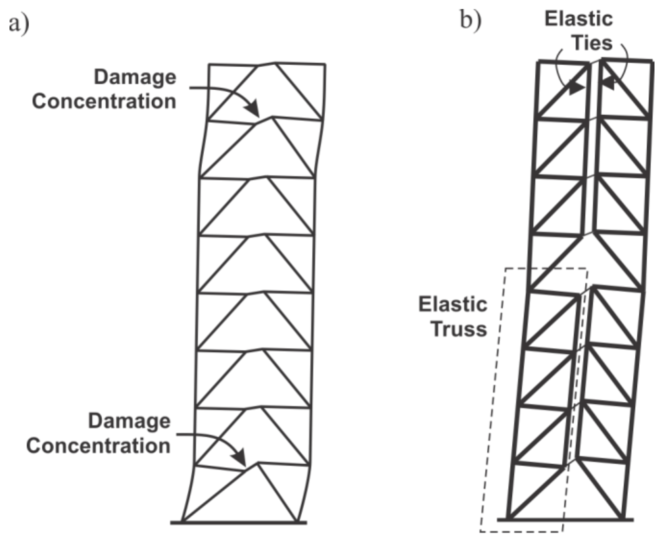

Figure 2. Deformed shape of an 8-storey structure: a) Conventional EBF; b) SESBF with 2 four-storey trussed spine segments.

For tall EBF buildings located in regions with low or moderate seismicity in eastern Canada, wind loading requirements may control link design over a portion of the building height. As a result, some of the links may have a higher capacity-to-seismic demand ratios compared to other links of the structure and this non-uniform seismic overstrength situation may lead to concentration of seismic damage in the floors exhibiting lower overstrength [16].

This paper presents a study that was conducted on a 24-storey steel braced frame building located in eastern Canada. The building was initially designed using an EBF system in accordance with current Canadian code provision. Wind load governed link design in the lower 80% of the building height. The building was then redesigned with the SESBF system to achieve a more uniformly distributed storey drift demand. Two SESBF configurations were studied. Nonlinear seismic time-history analyses were performed with a suite of 11 representative ground motion records to investigate and compare the seismic responses of the three braced frame structures.

BUILDING AND FRAMING SYSTEM STUDIED

Prototype building

A 24-storey prototype structure located on a site class E in Montreal, QC, was selected for this study. The building is a regular office building with the floor plan shown in Figure 3a. The design roof dead and snow loads are 3.4 kPa and 2.48 kPa, respectively. The respective dead load and live load supported by the floors are 4.5 kPa and 2.4 kPa. The weight of the exterior cladding is 1.2 kPa. As shown in Figure 3a, lateral loads in both orthogonal directions are resisted by four braced frames located along the exterior walls. The braced frames acting in the E-W direction are studied herein. As discussed, both the conventional EBF configuration and the SESBF configuration are studied. The two configurations are illustrated in Figure 3b. To increase the flexural stiffness of the braced frames at their bases, the width of both braced frame configurations is extended to two bays in the bottom 8 levels. For the SESBF system, two configurations were selected. The first SESBF configuration, SESBF-1, consists of three eight-storey spine segments that span over the entire building height. For the SESBF-2 configuration, only one elastic truss segment was designed over the top 8 storeys of the building. As will be discussed later, wind governed the design of the links in the bottom 20 storeys of the structure, and seismic loads only controlled the design in the remaining top four storeys. The elastic spine of the SESBF-2 structure therefore spans over the four seismically governed storeys and extends down in four storeys that are governed by wind loading. With this approach, it is expected that the elastic spine will provide a continuous elastic medium in the portion where the link seismic overstrength varies from a value of approximately 1.0 to higher values as wind design requirements start to control link sections.

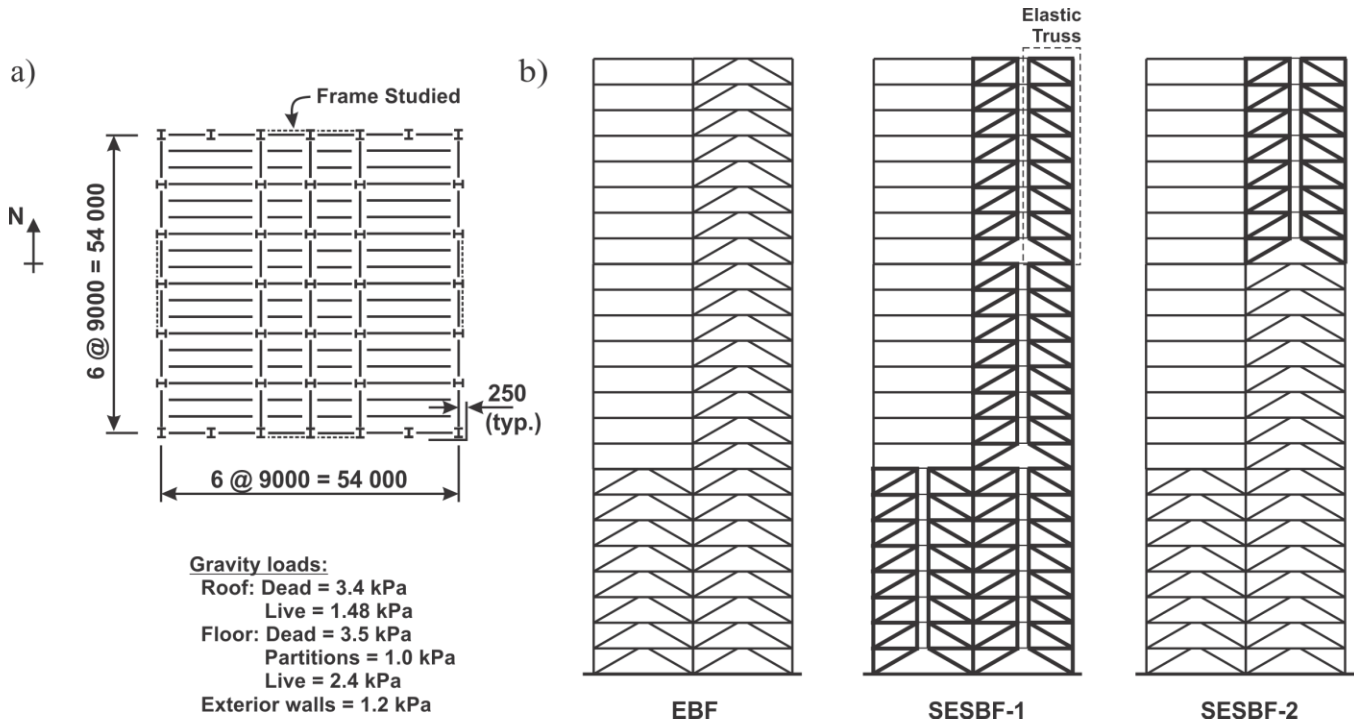

Figure 3 Floor plane view and elevation view of the lateral force resisting system: a) floor plane view and loading condition; b) structural configurations studied.

Design of the structures

Structure design was performed in accordance with the provisions of the 2015 NBCC [17] and the CSA S16-14 [18] design standard for steel structures. The braced frames were designed to resist both seismic and wind loads. Lateral deflections of the structures under both loading conditions were also verified against the limits specified in the design documents. In this study, the frames were first designed for earthquake resistance and then verified and corrected as necessary to satisfy wind loading limit states. In the NBCC, the seismic design base shear V is determined as:

$$V = \frac{S(T_a)M_V I_E W}{R_d R_o} \tag{1}$$

where S is the design spectrum, T_a is the fundamental period of the structure, M_v is the higher mode factor, I_E is the earthquake importance factor, W is the seismic weight and R_d and R_o are the ductility- and overstrength-related force modification factors, respectively. For the site studied, the specified values of S at periods of 0.2, 0.5, 1.0, 2.0, 5.0 and 10.0 s are respectively equal to 0.622g, 0.458g, 0.257g, 0.130g, 0.038g and 0.012g. For intermediate periods, S is linearly interpolated between these values. For the 24-storey structure studied, M_v = 1.0 and I_E = 1.0. For all three braced frame configurations, the values of R_d and R_o are selected as 4.0 and 1.5, as specified in the NBCC for steel ductile eccentrically braced frames. The NBCC allows using of the computed fundamental period of the structure for the design period T_a, provided that it does exceed 0.05 h_n for steel braced frames, where h_n is the building height in meters. The upper limits on T_a for base shear calculation is 4.56 s for the building studied. The final periods obtained from the model analysis for the first three vibration modes are given in Table 1. Because T_1 in all cases exceeded the upper limit for T_a, the upper limit had to used to determine the base shear. In addition, NBCC requires that V be not less than the value determined with T_a = 2.0 s, and this minimum force level applied to the three structures studied. For all frames, response spectrum analysis (RSA) was used to determine the seismic induced storey shears. As per NBCC, when RSA is used, the analysis results must be scaled up by the ratio $0.8V/V_d$ when V_d is less than $0.8V$, where V_d is the dynamic base shear obtained from RSA. The storey shears obtained from RSA are compared with those resulted from ESFP in Figure 4a. In the figure, the base shear obtained from RSA is scaled to $0.8V$. NBCC and CSA S16 also require that the storey shears obtained from RSA be further amplified by the U_2 factor to account for P-Δ effects. The U_2 factor in each storey is calculated as:

$$U_2 = \frac{1}{1 - \frac{\sum C_f \Delta_f}{\sum V_f h_s}} \tag{2}$$

where $\sum C_f$, Δ_f, V_f, h_s are the factored gravity loads, seismic deflection, total storey shear, and storey height at the storey, respectively. In this study, the U_2 factor was applied only for the EBF system because the SESBF system is expected to offer a stable inelastic seismic response. For the EBF structure, the U_2 factor varied from 1.17 at the base of the structure to 1.05 at the roof level, with a maximum of 1.34 at the 16^{th} level. As permitted in the NBCC, the factor was not applied in storeys where its value was less than 1.1. In Figure 4a, seismic induced link shears without and with the U_2 factor are presented. In the bottom 8-storeys, the values are reduced because the braced frame includes two braced bays.

Table 1. Periods of the first 3 vibration modes and base shear of the building

	T_a	V	V_d	T_1	T_2	T_3
	(s)	(%W)	(%W)	(s)	(s)	(s)
EBF	4.56	2.17	1.74	6.73	2.23	1.24
SESBF-1	4.56	2.17	1.74	6.96	2.18	1.19
SESBF-2	4.56	2.17	1.74	6.82	2.05	1.13

For wind loading, the 24-storey building in this study classifies as very dynamically sensitive because its lowest natural frequency is less than 0.25 Hz. Consequently, according to NBCC, the Wind Tunnel Procedure would need to be applied for determining wind loads for this structure. However, for the purpose of this study, the Dynamic analysis procedure was used for the calculation of wind loads. In that procedure, the gust factor depends on the dynamic properties of the structure. For this building, it was equal to 3.08 and 2.85 for ultimate and serviceability limit states, respectively. The reference wind pressures for the site are 0.42 kPa and 0.33 kPa for return periods of 50 and 10 years, respectively. The exposure coefficients were determined assuming a rough terrain condition and the topographical factor was taken equal to 1.0. In the calculations, a critical damping ratio of 1% was considered. In addition, second-order analysis under combined factored gravity and wind loads was performed to account for P-Δ effects. In Figure 4a, design link shears due to factored wind loading including P-Δ effects for the EBF structures are compared to the design link shears due to seismic loads.

As shown, design seismic storey shears only govern in the upper four storeys of the EBF. The design storey shears from wind loading governed in the rest of the structure. This was also the case for the SESBF-1 and SESBF-2 structures. As mentioned above, this condition was the motivation for the selection of an 8-storey spine segment in the upper part of the structure, i.e. having an elastic trussed spine extending over 4-storeys governed by seismic loads and four storeys below governed by wind loads. For the SESBF-1 structure, two other 8-storey trussed spines were included in the lower 16 storeys. For the EBF structure, the links at every level were designed individually to resist the governing design factored link shear forces. For SESBFs designed for seismic applications, the links in each segment are designed to resist the average seismic induced link shear force over the segment to encourage uniform link yielding in the segment under strong seismic demand. This approach could not be applied in this study because yielding of links is not permitted under factored wind load effects. Hence, each link of the SESBFs was also designed individually for its governing factored link shear force, as was done for the EBF structure. In the design, attention was paid to tightly size the links to match their resistances and design forces. For all three structures, the ratio between the factored shear resistance of the selected links and the design factored link shears lied between 0.97 to 1.07.

The vertical distribution of the link seismic overstrength is presented in Figure 4b to illustrate the effect of wind loading on the link resistance. The link seismic overstrength is defined as the ratio between the factored shear resistance of the selected links and the design factored link shears due to seismic loads. For the EBF, the latter included the amplification for P-Δ effects (U_2 factor). As shown, the link seismic overstrength is small in the top four storeys governed by seismic loads. In the storeys below, the link seismic overstrength increases significantly to reach values varying between 1.5 and 3.0 depending on the depending on the relative amplitude of the wind and seismic induced factored link shears along the height of the three structures.

For the EBF structure, all other structural members were designed to resist gravity induced loads plus lateral load effects corresponding to the probable resistance of the links, as specified in the CSA S16 standard. The same approach was used for the SESBF structures except that all members were also designed to resist the additional axial forces induced by the flexural response of the elastic spines. For all three braced frames, the wind induced storey drifts exceeded the limit of 1/500 h_s prescribed in the NBCC. Column and brace members were increased as necessary to reduce the wind deflections within the prescribed limits. The maximum seismic inter-storey drifts obtained from RSA were less than 1.5%h_s for all three braced frame configurations.

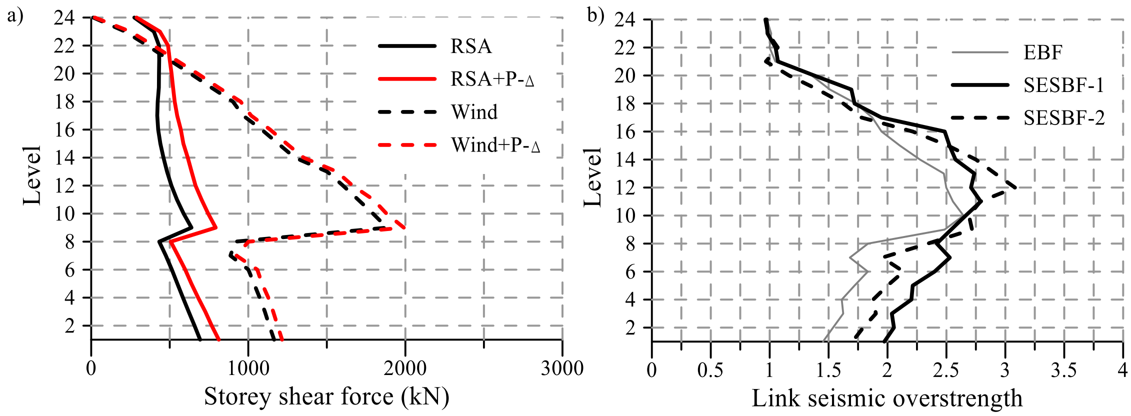

Figure 4 a) EBF design factored storey shears (/braced bay); b) Link seismic overstrength

SEISMIC RESPONSE

Numerical Model

Structural models of the braced frames were developed with the OpenSees platform [19]. The nonlinear response history analyses were performed on 2D models of one of the two E-W braced frames. Braces, columns, ties and beam segments outside of the links were modelled with force-based nonlinear beam-column elements with fiber discretization of the cross-section and initial imperfection to explicitly reproduce failure due to member yielding or inelastic buckling, both in-plane or out-of-plane. The Steel02 material that accounts for kinematic and isotropic hardening behaviour was assigned to these elements. The ductile links were modelled with a nonlinear zeroLength element capable of reproducing the hysteretic response of links yielding in shear. The element was coupled with the MinMax material to predict link failure and subsequent strength degradation resulting from excessive plastic rotation. This shear link model was calibrated against past experimental results [14].

In the model, Rayleigh damping corresponding to 3% of critical was assigned in the first and fifth modes of the structures. Initial stiffness proportional damping was assigned to all frame members except the zeroLength elements used in the shear links. Corotational transformation was used to account for geometric nonlinearities in the analyses. All three models included a P-Δ column carrying the concomitant gravity loads consisting of the dead load plus 50% of the live load and 25% of the roof snow load supported by the gravity columns of the buildings laterally stabilized by the braced frame. The P-Δ column was modelled using elastic beam elements with flexural stiffness equal to the sum of the flexural stiffness of the gravity columns it represented. The P-Δ column was continuous with the same properties over two consecutive storeys, and pinned splices were assumed at every second floor level.

Selection and scaling of ground motion records

The structures were subjected to an ensemble of 11 representative simulated ground motion records. The ensemble comprised a suite of 5 records from M6 earthquakes from 13 to 21 km for the short and moderate period range and one suite of 6 records from M7 earthquakes from 17-100 km for the long period range, as recommended by Atkinson [20]. The records were selected from the eastern Canada sets of simulated ground motion time histories for site class E available on the Seismic Engineering Toolbox website (https://www.seismotoolbox.ca). Selection and scaling of the ground motion records was performed in accordance with the guidelines provided in the Commentary J of NBCC 2015. The scenario-specific period range T_{RS} for the suite of M6 records was from 0.1 to 1.0 s. For the M7 records, T_{RS} was set from 0.5 to 10 s. The records of each suite were scaled individually so that their spectra matched, on average, the target response spectrum over the scenario-specific period range T_{RS} for the suite. Additional scaling was then performed to ensure that the mean spectrum of each suite was no less than 90% of the target spectrum over the suite period range T_{RS}. The spectra of the scaled ground motion records of each suite are presented in Figure 5. As shown, the two suites of records, when combined together, could properly represent the design seismic conditions for the site considered.

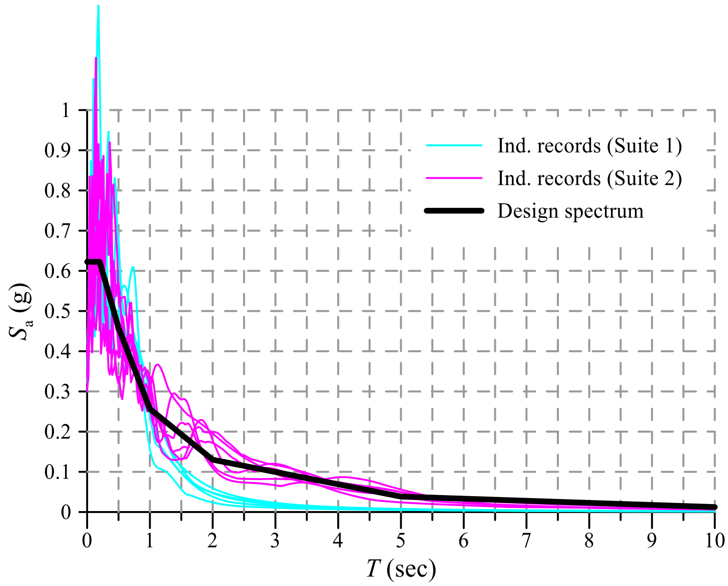

Figure 5 5% damped acceleration spectra of the scaled ground motion records.

Analysis results and discussions

According to NBCC, when 11 ground motion records are used, the design seismic demand from the NLRHA was computed as the mean value of the largest 5 results, and a structure was said to have an unacceptable response when collapse was observed in one ground motion. Envelopes of the peak storey drifts and link plastic rotations from the individual ground motions are presented for the EBF structure in Figure 6. As shown, the M6 earthquake records with short dominant periods had limited effects on the structure compared to the M7 earthquake records having greater energy in the long period range. As expected, the abrupt variation in link seismic overstrength in the upper levels resulted in large storey drift and link plastic rotation demands in the 18th to 22th levels. The structure became dynamically unstable at these levels in three of the 11 analyses, and peak storey drifts in excess of the NBCC limit of 2.5% h_s with link failures were observed under two other ground motions. Conversely, the EBF structure experienced little to no structural damage in the bottom floors, with peak storey drifts remaining below 1% h_s. This unacceptable EBF seismic response is attributed to the non-uniform link seismic overstrength present in the structure and the limited capacity of EBFs for vertically distributing the seismic induced inelastic demand.

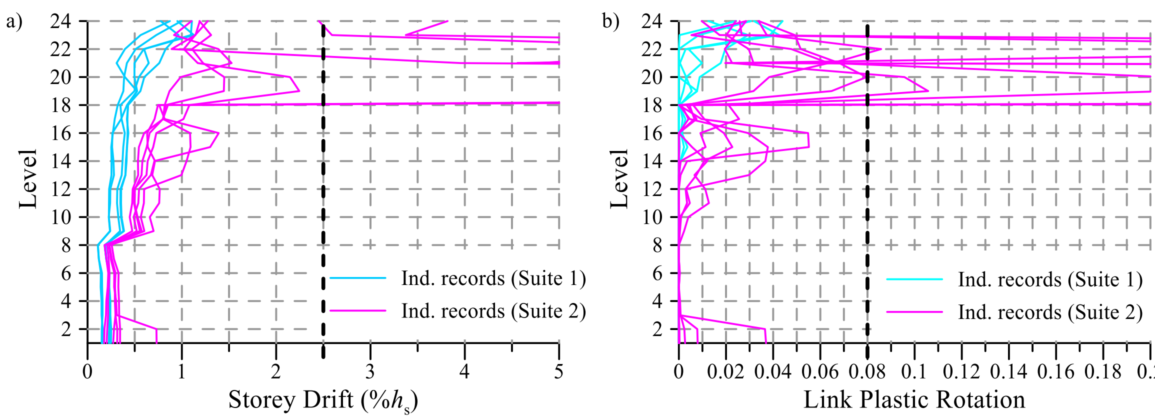

Figure 6 Peak storey drift profiles and peak link inelastic rotations for the conventional EBF: a) Peak storey drift profiles; b) Peak link inelastic rotation profiles.

Peak storey drift and link plastic rotation demands for the SESBF-1 and SESBF-2 structures are presented in Figures 7 and 8, respectively. Both structures offered stable seismic response under all ground motions. They sustained comparable and more uniform peak lateral displacements, with design seismic values of the storey drift well below the 2.5% h_s NBCC limit. In all links of the two structures, seismic design values of the plastic rotations also remained lower than the 0.08 limit prescribed in the CSA S16 standard. The positive impact of the elastic trussed spines on the vertical distribution of the storey drifts and link plastic rotations can be readily seen in the figures. For the SESBF-2 structure, excessive inelastic link deformations can be observed under one ground motion at the 13th and 14th floor levels, two levels below the 8-storey elastic spine provided in the upper part of the structure. This response was mitigated in the SESBF-1 structure that included 8-storey elastic spines over its full height. However, the results show that, globally, minimal gain was achieved with the two additional elastic spines located in the wind load governed region of the SESBF-1 structure. This suggests that solutions involving elastic spines strategically located along the structure height can be sufficient to annihilate the adverse effects from non-uniform seismic overstrength.

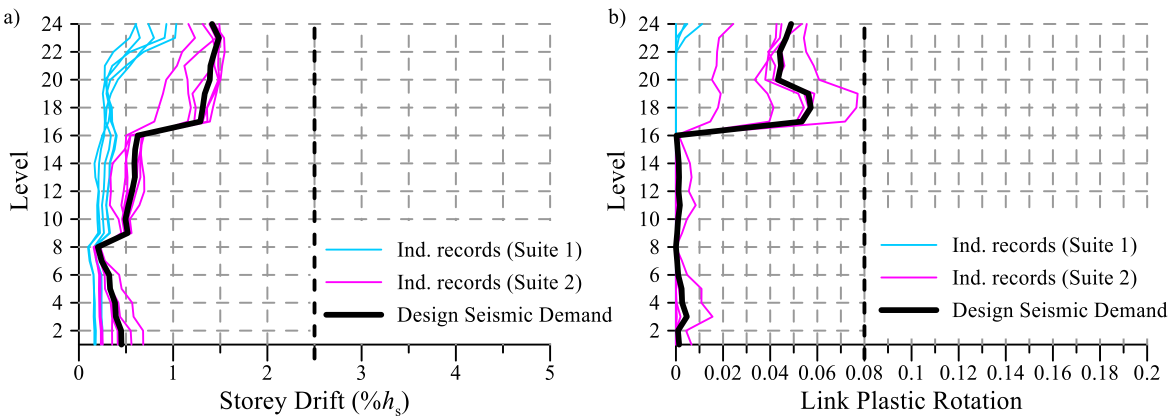

Figure 7 Peak storey drift profiles and peak link inelastic rotations for the SESBF-1: a) Peak storey drift profiles; b) Peak link inelastic rotation profiles.

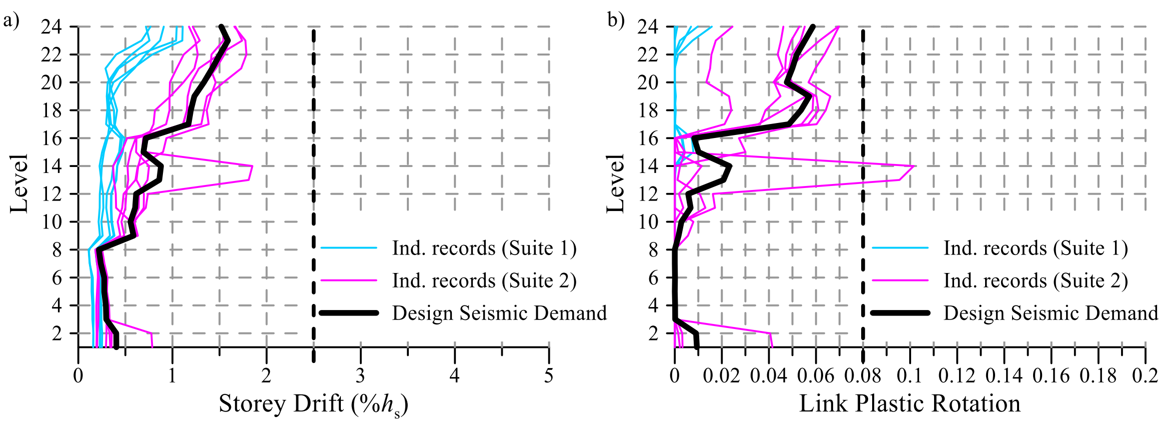

Figure 8 Peak storey drift profiles and peak link inelastic rotations for the SESBF-2: a) Peak storey drift profiles; b) Peak link inelastic rotation profiles.

CONCLUSIONS

A study was performed to assess the impact of non-uniform seismic overstrength resulting from wind loading conditions on the seismic response of tall multi-storey steel braced frame structures. The study was performed for a 24-storey office building located on a class E site in Montreal, QC. The building seismic force resisting system consisted of eccentrically braced frames located along the exterior walls. The structure was designed in accordance with the Canadian code seismic and wind provisions. The design of the EBF links was governed by factored wind loads in the first 20 levels; seismic loads governed in the remaining storeys. This resulted in link seismic overstrength varying from 1.0 in the uppermost four storeys to a maximum of 2.6 in the intermediate levels. The segmental elastic spine braced frame (SESBF) system was investigated to prevent concentration of seismic inelastic demand in the structure. Two SESBF configurations were studied: SESBF-1 with three eight-storey spine segments over the full building height and SESBF-2 with one eight-storey segment located in the top eight levels. The seismic response of the structures was investigated through nonlinear response history analysis performed with site representative earthquake ground motion records. The following conclusions can be drawn from this study:

- Because of the non-uniform link seismic overstrength, the conventional EBF structure exhibited unacceptable seismic response characterized by excessive inelastic deformations and structural collapse in the upper levels where seismic loading had governed the design of the links.
- Both SESBF configurations were found to be effective for mitigating concentrations of inelastic deformations caused by the uneven lateral seismic overstrength of the structure.

- The satisfactory response obtained with the SESBF-2 configuration suggests that elastic trussed spines spanning over the storeys where link design is governed by seismic loads and extending over a few storeys in the storeys where wind controls link design can be sufficient to achieve acceptable seismic response. Further studies are needed to confirm this assumption for other buildings and develop appropriate design guidance for this strategy.

This limited study revealed that building structures exhibiting non-uniform seismic overstrength resulting from different governing loading conditions or limit states can exhibit unacceptable seismic response, even when designed in accordance with current seismic provisions of the NBCC. Pronounced variations in lateral seismic overstrength over the structure height should be treated as a vertical structural irregularity in seismic design, and special provisions should be developed to mitigate its detrimental effects on building seismic response.

ACKNOWLEDGMENTS

Financial support for this project was provided by the Natural Sciences and Engineering Research Council of Canada and the Fonds de Recherche en Nature et Technologies (FRQNT) of the Government of Québec.

REFERENCES

[1] Martini, K., Amin, N., Lee, P.L., and Bonowitz, D. (1990). "The potential role of nonlinear analysis in the seismic design of building structures." *Proceedings 4th National Conference on Earthquake Engineering*, Palm Springs, CA, 2, 67-76.

[2] Ricles, J.M., and Bolin, S.M. (1990). "Energy dissipation in eccentrically braced frames." *Proceedings 4th National Conference on Earthquake Engineering.*, Palm Springs, CA, 2, 309-318.

[3] Chao, S.-H., and Goel, S. (2005). *Performance-based seismic design of eccentrically braced frames using target drift and yield mechanism as performance criteria*. Report UMCEE-05-05, University of Michigan, Ann Arbor, MI.

[4] Richards, P. W. and Uang, C. M. (2006). "Testing protocol for short links in eccentrically braced frames". *Journal of Structural Engineering*, ASCE, 132(8), 1183-91.

[5] Bosco, M., and Rossi, P.P. (2009). "Seismic performance of eccentrically steel braced frames," *Engineering Structures*, 31, 664-679.

[6] Koboevic, S., David, S.O. (2010). Design and seismic behaviour of taller eccentrically braced frames. *Canadian Journal of Civil Engineering*, 37(2), 195-208.

[7] Koboevic, S., Rozon, J., and Tremblay, R. (2012). Seismic performance of low-to-moderate height eccentrically braced steel frames designed for North-American seismic conditions. *Journal Structural Engineering*, 138(12), 1465-1476.

[8] Popov, E.P., Ricles, J.M., and Kasai, K. (1992). "Methodology for optimum EBF link design." *Proc. 10th World Conf. on Earthquake Engineering*, 7, 3983-3988.

[9] Whittaker, A., Uang, C.-M., and Bertero, V.V. (1990). *An Experimental Study of the Behaviour of Dual Steel Systems*. Report no. UCB/EERC-88/14, Earthquake Engineering Research Center, University of California, Berkeley, CA.

[10] Ghersi, A., Neri, F., Rossi, P.P., and Perretti, A. (2000). "Seismic response of tied and trussed eccentrically braced frames." *Proceedings Stessa 2000 Conference,* Montreal, Canada, 495-502.

[11] Ghersi, A., Pantano, S., and Rossi, P.P. (2003). "On the design of tied braced frames." *Proceedings Stessa 2003 Conference,* Naples, Italy, 413-429.

[12] Ghersi, A., Marino, E.M., and Rossi, P.P., (2006). "Innovative design criteria, based on ductility control for earthquake resistant steel structures." in *Innovative Steel Structures for Seismic Protection of Buildings*, Monza, Italy: Polimetrica International Scientific Publisher, 155-207.

[13] Rossi, P.P. (2007). "A design procedure for tied braced frames." *Earthquake Engineering and Structural Dynamics*, 36, 2227–2248.

[14] Chen, L., Tremblay, R. and Tirca, L. (2019a). "Modular tied eccentrically braced frames for improved seismic response of tall buildings". *Journal of Constructional Steel Research.*, Vol. 155, pp. 370-384.

[15] Chen, L., Tremblay, R., and Tirca, L. (2019b). "Practical Seismic Design Procedure for Tall Steel Braced Frames with Segmental Elastic Spines." *Journal of Constructional Steel Research*, Vol. 153, pp. 395-415.

[16] Faucher, L.A. (2017). "Conception et performances sismiques des contreventements excentriques avec liens modulaires." Master thesis, Department of geological, civil and mining engineering, Ecole Polytechnique de Montreal, Montreal, QC. (In French)

[17] NRCC. (2015). *National Building Code of Canada*, 13th ed., National Research Council of Canada, Ottawa, ON.

[18] CSA. (2014). *CSA S16-14, Design of Steel Structures*. Canadian Standards Association (CSA), Toronto, ON.

[19] McKenna, F. and Fenves, G.L. 2004. *Open System for Earthquake Engineering Simulation (OpenSees)*. Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, CA.

[20] Atkinson, G.M. (2009). "Earthquake Time Histories Compatible with the 2005 NBCC Uniform Hazard Spectrum." Canadian Journal of Civil Engineering, 36, 991-1000.