Study of global seismic response of eccentrically braced frames with long links

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ABSTRACT: The global seismic response of 3- and 8-storey eccentrically braced frames (EBFs) with long, flexure-critical links is studied. In spite of lesser ductility, EBFs with long links offer more flexible architectural arrangements, which can be advantageous for new buildings and for seismic retrofits. However, high axial loads and bending moments introduced by yielding links into outer beam segments often require strengthening of the beam which can be avoided if some yielding is admitted in these elements. Limited evidence is available regarding the possible extent of this yielding and the impact of this behaviour on the overall frame response. The study was carried out for chevron-type frames designed for typical western and eastern North-American seismic conditions. Attention was directed in particular to the behaviour of the links and the outer beam segments. The model for nonlinear time history analysis was built using the OpenSees computer program. Links were represented by elastic beam elements with one zero-length rotational spring attached at the ends of each link. The inelastic spring behaviour was described using Steel02 material. The link model was calibrated against experimental results on long links available in literature. The outer beams, braces and columns were modeled using nonlinear beam-column elements with fiber discretization of the cross section to reproduce cross-sectional yielding as well as in- and out-of-plane flexural buckling. The results show that the amplification factors applied on link forces to determine design forces for other frame members appear conservative, particularly for eastern location characterized by high-frequency ground motions. The results suggest that the flexural yielding of outer beam segments is acceptable if combined with the flexural strength of the brace and beam is at least equal to the link end moment, and if the brace can resist the imposed axial-moment demand.

KEY WORDS: Seismic design; Nonlinear time history analysis; Global structural behavior; Eccentrically braced frames; Long links; Seismic response of steel structures.

1 INTRODUCTION

Eccentrically braced frames combine high strength, stiffness and ductility and represent a very efficient traditional seismic load resisting system in steel buildings. Chevron-type configuration is usually preferred because it avoids a beam-to-column connection in the link region. Short links are commonly employed to increase system ductility developed through stable and efficient shear yielding. However, in spite of lesser ductility, EBFs with long links are gaining popularity among designers as they offer more flexible architectural arrangements and can accommodate larger openings. These features are advantageous for new buildings structures and for seismic retrofits.

Design of outer beam segments in EBFs with long links is challenging. Yielded and strain hardened links introduce high axial forces and bending moments in these elements, and it is very difficult to maintain a uniform beam section unless some yielding is accepted. In general, the occurrence of limited yielding of outer beam segments is not considered detrimental to satisfactory frame behaviour as long as lateral stability of the beam is provided [1]. However, limited evidence is available regarding the possible extent of this yielding and the impact of this behaviour on overall frame response. Consequently, inelastic behaviour of the outer beam segments is not integrated into current seismic design procedures in North America [2,3]. Allowing outer beam yielding in design could avoid beam strengthening that may otherwise be necessary. A study conducted by Koboevic et al. [4] on EBFs with shear-critical links and a moment-resistant brace-to-beam connection confirmed that the rotational demand in the outer beam segments was not excessive and can be easily accommodated. This confirmed that for the frames with shear and intermediate links, yielding of laterally stable outer beam segments is acceptable, provided that the braces have sufficient stiffness and strength to resist part of the imposed end link moment. For EBFs with long links, an imposed rotational demand on the outer beam segment could be more significant and thus further investigation is necessary to establish if outer beam yielding can be safely incorporated in the design for such systems.

This paper presents the study carried out for 3- and 8-storey chevron-type EBFs with long, flexural links. Frames were designed for typical western and eastern North-American seismic conditions, and their seismic response was examined using nonlinear time history analysis for selected ground motions. Attention was directed in particular to the behaviour of the links and the outer beam segments. The analytical model was built using the OpenSees computer program. Link response was tracked through maximum link forces and inelastic link rotations. The results were compared to the design predictions. The response of outer beam segments was observed by tracking the moment distributions, the frequency and the extent of yielding and inelastic beam rotations. The sensitivity of the structural response to the variability of
ground motion input and the selection and scaling procedures is also investigated.

2 BUILDING DESIGN

3- and 8-storey buildings were designed for two Canadian locations, Montreal, QC (MTL3 and MTL8), and Vancouver, BC (VCR3 and VCR8) assuming Class C site conditions (360m/s ≤ v_s ≤ 760m/s) [5,6]. The typical floor plan is illustrated in Figure 1. Two chevron-type EBFs with flexure-critical links provided seismic resistance in the north-south direction. Contrary to what is illustrated in Figure 1, the columns were oriented to bend about the weak-axis in order to ensure consistent numerical modeling. Perimeter moment-resisting frames resisted lateral loads in perpendicular direction. The braced bay width, L, was 9 m while the typical storey height was 3.5 m with 4 m selected at the first storey. Based on the recommendations by Rossi and Lombardo [7] and Ozendekci [8], the link length e = 2.5m was selected resulting in e/L ratio equal to about 0.3. Brace-to-beam connections were considered moment-resistant, while all other connections were assumed pinned.

Figure 2. Design gravity loads and typical floor plan of the studied buildings

The structures were designed according to the 2010 National Building Code of Canada [9]. The design gravity loads are given in Figure 2. Initially, the seismic base shear, V_s, was determined according to NBCC 2010 equivalent static force procedure, and served as a base to calibrate the results of the subsequent response spectrum analysis as required by the Canadian building code.

\[ V = \frac{S(T_a)M_s J_e W}{R_d R_o} \]  

(1)

where \( T_a \) is the empirical structural period, \( T_s = 0.025h_n \) being a total height of the structure, \( S(T) \) is the spectral acceleration at the period based on the probability of exceedance of 2 percent in 50 years and modified by foundation coefficients \( F_s \) and \( F_v \) to reflect the soil conditions at the design site, \( M_s \) is the factor accounting for the increase in base shear due to the higher mode effect; \( I_e \) is the structure importance factor; \( W \) is the total seismic weight tributary to the frame; \( R_d \) is the ductility-related force reduction factor and \( R_o \) is the overstrength-related force reduction factor. In this study, \( R_d = 4.0 \); \( R_o = 1.5 \); \( F_s = F_v = 1.0 \). The design base shear was determined using the increased period equal to 2.0 \( T_s \) for 3- and 8-storey buildings, as permitted by NBCC 2010. The base shear \( V \) calculated using equation [1], needs not exceed 2/3 the value of \( V_s \) with \( T_s = 0.2 \) s, and must be greater than the value obtained with \( T_s = 2.0 \) s. Accidental torsion was not considered in the design to ensure compatibility with the 2D non-linear analysis. Table 1 summarizes design base shear calculations. Wind loads did not govern the design of the Vancouver building, but were critical for several storeys in the MTL8 building.

Table 1. Seismic design base shear calculations

<table>
<thead>
<tr>
<th>Frame</th>
<th>S(T) (g)</th>
<th>Seismic weight (kN)</th>
<th>Base shear (V) (kN)</th>
<th>V/W (%)</th>
<th>Computed period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MTL3</td>
<td>0.293</td>
<td>20363</td>
<td>994</td>
<td>4.88</td>
<td>1.3</td>
</tr>
<tr>
<td>VCR3</td>
<td>0.691</td>
<td>19991</td>
<td>2062</td>
<td>10.32</td>
<td>0.8</td>
</tr>
<tr>
<td>MTL8</td>
<td>0.111</td>
<td>61243</td>
<td>1134</td>
<td>1.85</td>
<td>3.0</td>
</tr>
<tr>
<td>VCR8</td>
<td>0.268</td>
<td>60869</td>
<td>2716</td>
<td>4.46</td>
<td>1.7</td>
</tr>
</tbody>
</table>

The EBF frame design was conducted in compliance with the CAN/CSA S16-09 seismic provisions [2]. Beams and columns were selected from W shapes, and braces were chosen from available square tubular (HSS) sections and W sections. Note that the W brace sections were oriented for bending about the weak axis to achieve consistency with the modelling which led to heavier brace sections. All members were made from CSA-G40.21-350W steel with \( F_y = 350 \) MPa. In the capacity design phase, link sections with adequate length, class of the section and factored seismic resistance, \( V_s \), were selected to resist the factored seismic loads. For long, centrally located links which yield in flexure and have zero axial load, \( V_s = 2\phi M_p/e \), where \( \phi \) is the resistance factor (= 0.9 for steel) and \( M_p \) is the nominal plastic bending resistance (\( M_p = Z F_y \)). All selected links qualify as long links except the first floor link in the VCR8 structure which qualifies as intermediary. This particularity was subsequently addressed in the numerical model.

The braces and the outer beams were designed as beam-columns to resist the forces introduced by 1.3 \( R_yF_y \) times the nominal resistance of the links, \( V_s \), where \( R_y \) is the expected-to-nominal yield strength ratio (\( R_yF_y = 385 \) MPa). The link end bending moments were initially distributed between the brace and the outer beam in proportion to their relative flexural stiffness until the portion of the bending moment assigned to the outer beam segment surpassed its resistance. The remaining bending moment was transmitted to the brace. This design strategy assumes that yielding of the outer beam is acceptable. After the braces are selected, the flexural rigidities of the braces and beams are recalculated to verify that the initially assumed moment distribution still applies.

The columns were considered continuous over the height and tiered in two-storey segments. Axial forces due to the gravity loads were combined with the forces introduced by yielding links (1.3 \( R_yV_p \) and 1.15 \( R_yV_p \) for the top and lower tiers respectively). Additional bending moments resulting from column continuity and the relative storey movements were considered in the design, as prescribed by CSA S16. Excessive link rotations imposed some section modifications in the VCR8 frame. Inter-storey drift requirements did not control the design of any of the frames.
studied. Once the ductility design completed, the frames were verified for adequate stiffness and strength under all relevant load combinations including gravity loads, notional loads, wind and seismic loads. Overall, in spite of the larger brace sections, the weight of the structures was smaller compared to the alternative design in which an elastic response of the outer beam segments was required. Final periods obtained from the analysis by adding three simple pin-ended columns representing the corner, interior and exterior gravity columns. The outer beam segments, braces and columns were modeled using eight nonlinear beam-column elements with fiber discretization of the cross section to reproduce cross-sectional yielding as well as in- and out-of plane flexural buckling. A detailed description is available in Koboevic et al. [4]. Initial member out-of-straightness was specified for these elements. Each element included 4 integration points, and a total of 16 fibers were used to model the cross-section. Rotational spring elements were incorporated at the brace ends to account for the restraint conditions induced by the gusset plates. For the columns, the number of fibres was increased to 50 and the Steel02 material was modified to account for residual stresses. Note that in the design, the W brace sections and the columns were oriented so that the bending was induced about the weak section axis. In this case, failure could not be governed by lateral-torsional buckling, and thus the analytical model was appropriate.

Gravity loads corresponding to 100% dead load, 50% floor live load and 25% roof snow load were applied prior to the seismic analysis. The seismic masses were concentrated at the beam-to-column joints. P-Δ effects were included in the analysis by adding three simple pin-ended columns representing the corner, interior and exterior gravity columns. These columns carried one half of the total gravity loads of the building, excluding the tributary gravity loads supported directly by the EBF.

3 MODELLING FOR NONLINEAR TIME HISTORY ANALYSIS

3.1 OpenSees model

A numerical model was implemented on the OpenSees platform [10]. To represent the link behavior Bilin material [11] was initially used. This material incorporates the modified Ibarra-Medina-Krawinkler deterioration model with bilinear hysteretic response and was successfully used to study inelastic response of beams in moment resisting frames. However, in this study it was not possible to achieve a good agreement with experimental results as reported by Okazaki et al. [12,13] for two long link specimens.

The model retained for this study is the adaptation of the one used by Koboevic et al. [4] to examine seismic response of EBFs with short links. The central portion of the link is represented by an elastic beam element [14]. At each end of the elastic beam, a single spring is attached. The inelastic behaviour of the spring is described using the Giuffré-Menegotto-Pinto (Steel02) hysteretic material. The following parameters for Steel02 material were determined from a calibration using Okazaki’s test data: \( R_0 = 30, cR_1 = 0.925, cR_2 = 0.15, b = 0.00438, a_1 = a_3 = 0.4, \) and \( a_2 = a_4 = 22. \) This approach permitted to represent the behaviour of long links and intermediate links using the same model, and showed a good match with experimental results.

The outer beam segments, braces and columns were modeled using eight nonlinear beam-column elements with fiber discretization of the cross section to reproduce cross-sectional yielding as well as in- and out-of plane flexural buckling. A detailed description is available in Koboevic et al. [4]. Initial member out-of-straightness was specified for these elements. Each element included 4 integration points, and a total of 16 fibers were used to model the cross-section. Rotational spring elements were incorporated at the brace ends to account for the restraint conditions induced by the gusset plates. For the columns, the number of fibres was increased to 50 and the Steel02 material was modified to account for residual stresses. Note that in the design, the W brace sections and the columns were oriented so that the bending was induced about the weak section axis. In this case, failure could not be governed by lateral-torsional buckling, and thus the analytical model was appropriate.

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3.2 Selection of ground motions

Nonlinear time history analyses were carried out for the sets of ground motion records compatible with NBCC design spectra for the two studied locations. Special attention was devoted to the selection of earthquake records and calibration procedures. Initially, the M-R scenarios that contribute most significantly to seismic hazard in Vancouver and Montreal were identified (Vancouver: \( M_{6.5} \) at 30 km and \( M_{7.5} \) at 70 km; Montreal: \( M_{6.0} \) at 15 km and 20 km; \( M_{7.0} \) at 50 km), and the ground motion records corresponding to these scenarios were pre-selected.

Two methods were then used for the final selection and calibration, namely (i) the Least-Moving-Average method described by Morteza et al. [15], and (ii) the method developed by Atkinson [16]. The first set, identified in the further text by LMA, is identical to that studied by Morteza et al. and consisted of 20 historical recording for Vancouver, and 10 hybrid and 10 simulated records for Montreal. The use of synthetic records was motivated by the lack of historical ground motion records from large earthquakes in eastern Canada, typically rich in high frequencies. The second set, denoted by ATK included 13 simulated records for Vancouver and 10 for Montreal. For this set only simulated records were considered to maintain consistency with the proposed selection and scaling procedure described in Atkinson [16]. Details on this set can be found in [17]. Comparison between the mean acceleration spectrum of the scaled records and NBCC design spectrum for Vancouver is shown in Figures 2(a) and 3(b) for LMA and ATK methods, respectively.

![Figure 2. Design spectrum for Vancouver (NBCC 2010) and mean acceleration response spectra of scaled ground motion records](image-url)
4 DISCUSSION OF RESULTS

The structural response was examined by tracking the peak values of the shear forces and inelastic rotations in the links, the frequency and the duration of outer beam inelastic excursions, the distribution of the link end moment between the outer beam segment and the brace, and the inelastic rotational demand imposed on the outer beam segments. Results presented for outer beams segments are these obtained for the eight-storey frame in Vancouver (VCR8) because it was the only structure that experienced more significant inelastic response of these elements. For each acceleration record, peak values of the response parameters were found at every storey and the mean and 84th percentile values were calculated for each ground motion ensemble. The results were also used to validate the CAN/CSA S16-09 design requirements. Design predictions were assessed based on mean results.

4.1 Response of links

To allow direct comparison with the CSA S16-09 amplification factors used in design to account for link strain hardening, maximum link shear forces were normalised by the expected shear strength of the links, \( V_{pr} = R_v V_p \). Note that \( V_p \) is equal to \( 2M_p/e \), where \( M_p \) is the plastic moment of the link section. Figure 3 shows the results obtained for the eight-storey frame in Vancouver. The thicker lines represent mean results and the thinner lines show the 84th percentile values. Links in all storeys yielded in flexure but their contribution in energy dissipation was not uniform. Higher forces developed in the upper storeys, reaching the maximum mean value of 1.2 \( V_{pr} \) and 1.17\( V_{pr} \) for LMA and ATK records respectively. The mean and 84th percentile results were lower than the design predictions. Although the top two storey links yielded more often under LMA ground motions, the maximum values of link shear forces and their distributions were similar for the two ground motion sets.

In the MTL8 frame smaller link rotations were recorded compared to the VCR8 structure. Mean results were below the design limit for all links, but exceeded the limit at the 84th percentile level at the eighth storey (0.023 rad and 0.003 rad for ATK and LMA records respectively). Similar observation regarding link rotations were made for the three-storey frames: maximum link rotation occurred in the links which developed the largest force. At the mean level, link rotational limit was not exceeded while at the 84th percentile level the maximum rotations reached about 1.5 times the design limit. Overall LMA records imposed larger rotational demand and the differences in results obtained for two ground motion sets were significant. In previous studies conducted on EBFs with shear critical links [4], it was noted that the excessive peak link rotations were always accompanied by very small peak rotations in the other direction. Among the frames examined in this study, such behaviour was observed only for the VCR8 frame.

In the results for all frames show that, contrary to the force response, the deformation response is much more sensitive to the variability of ground motion input. The assessment of

\[ \text{Figure 2. Maximum normalised link shear forces for the VCR8 structure} \]

\[ \text{Figure 3. Maximum inelastic link rotations for VCR8 structure} \]
structural behaviour can therefore be significantly influenced by the selection and scaling procedures implemented to constitute ground motion sets for nonlinear time-history analysis.

### 4.2 Response of outer beam segments

To avoid the unnecessary increase of link section due to the high force demand imposed on the outer beam segments in the capacity design procedure, the outer beam segments in this study were designed assuming that yielding in these elements is acceptable. Brace-to-beam connections were considered moment-resistant, but rotation compatibility between the beams and the braces at the connection was not sought. The braces were selected such that the combined flexural strength of the brace and the beam outside the link, in the presence of the concomitant axial loads, surpassed the link end moment. It was of interest to see if yielding occurred in the outer beam segments as anticipated, and if the fractions of link end moment transferred to the outer beams and braces were well estimated in the design.

More frequent inelastic excursions of the beam segments outside of the links were observed under LMA records. Yielding was most pronounced at the 1st, 7th and 8th storeys, where more significant inelastic behaviour was observed in the links. Not all the records induced inelastic response in the outer beams, and the duration of the yielding was not excessive (1.13s for all LMA records).

In Table 2, design predictions for the outer beam and brace end moments are compared to mean results of nonlinear time-history analysis obtained for two ground motion sets. A very good match was observed. When outer beam segment responded elastically, distribution reflects the relative flexural rigidity between the two elements. When the yielding of the outer beam segment occurred, distribution reflects the increase in the brace moment transmitted to brace by the yielding beam.

<table>
<thead>
<tr>
<th>Storey</th>
<th>LMA/ATK Design</th>
<th>Outer beam</th>
<th>Brace</th>
<th>Outer beam</th>
<th>Brace</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>66/67</td>
<td>70</td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>58/59</td>
<td>60</td>
<td>40</td>
<td></td>
<td></td>
</tr>
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<td>58/59</td>
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<td>41</td>
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</tr>
<tr>
<td>2</td>
<td>48/50</td>
<td>55</td>
<td>45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>58/60</td>
<td>56</td>
<td>44</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2 summarises the calculations of rotational capacities for outer beam segments in the VCR8 frames. To determine the available inelastic rotational capacity, \( \theta_e \), following Kemps’s method, the rotational capacity, \( R \), is calculated and multiplied by the elastic rotation, \( \theta_e \). Elastic rotation is determined by assuming that the outer beam can be represented as a cantilever beam loaded at the free end point. According to the Okazaki method, the total maximal rotation for a given beam section is first determined, and the elastic rotation, \( \theta_e \), is then subtracted to obtain the available inelastic rotational capacity.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Kemp LMA/ATK</th>
<th>Okazaki LMA/ATK</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>43</td>
<td>111</td>
</tr>
<tr>
<td>7</td>
<td>39</td>
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<td>2</td>
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<td>2</td>
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<td>2</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>7</td>
</tr>
</tbody>
</table>

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given in Table 4 represent the mean values of demand-to-capacity ratios, expressed as a percentage of the available inelastic rotational capacity. The response of the outer beams in the first five stories was predominantly elastic for the two ground motion sets. The top two storey outer beams developed larger inelastic rotations, particularly for the LMA records. When Kemp limiting rotations are considered, the demand reaches up to 40 percent of the available rotation capacity which could easily be accommodated. However, when Okazaki limits are considered, the rotational demand recorded in the 7th storey reaches 70% of the available inelastic rotation and exceeds the maximum allowable rotation in the top storey beam. This demand may be considered excessive, however it is generally of short duration and the impact on global behavior is not necessarily detrimental. Experimental data related to EBF seismic response are limited to studies of link behaviour and no information is presently available on the acceptable response of the other frame members. Further experimental studies investigating the global EBF response are necessary to determine what level of inelastic outer beam rotations can be accepted without compromising the overall structural response.

Similarly to what was observed for the link deformation response, the inelastic outer beam rotations were highly sensitive to the variability of ground motion input. For the top storey beam, LMA ground motion records induced on average 1.4 times larger inelastic deformations compared to the ATK ground motion set. This confirms that the selection and scaling methods used to constitute ground motion suites for non-linear time-history analysis need to be carefully considered when assessing the seismic response of structures and validating design procedures.

5 CONCLUSIONS

3- and 8-storey eccentrically braced steel frames with flexural links were designed for typical western and eastern North American locations (Vancouver and Montreal) and their seismic response was investigated for selected ground motions representative of 2% in 50-year hazard level. Nonlinear time-history analyses were carried out on the OpenSees platform. Two sets of ground motions were selected to investigate the impact of ground motion selection and scaling procedures on the inelastic frame response. The contribution of concrete floor slabs to the frame lateral stiffness and strength was not considered, so the results reported in this paper are valid for chevron-type bare steel EBFS.

Allowing the yielding of the outer beam segments in the design avoided the unnecessary increase of link beam sections due to the demand imposed on the outer beam segments, and permitted to maintain the resistance of the links close to the demand. In turn, brace-to-beam connections had to be conceived as moment resistant and larger brace sections had to be selected to resist the excess link end moment transferred from the outer beam segment. However, in spite of the larger brace sections, the weight of the structures was smaller compared to alternative design in which an elastic response of the outer beam segments was required.

An analytical model was built using the OpenSees platform. Steel02 material offered a good compromise to represent the inelastic behaviour of long and intermediate links using the same model, and showed good agreement with experimental results available in literature. However, more accurate representation for the flexural links could possibly be achieved using Bilin’s material, and should be further studied. The OpenSees model permitted a more refined representation of the behaviour of frame members other than the links, including cross-section yielding and in-plane flexural buckling of the outer beam segments, as well as cross-sectional yielding and in-plane and out-of-plane flexural buckling of braces and columns. The impact of inelastic behaviour of the outer beam segments on the overall seismic performance of the frames could therefore be assessed.

Inspection of shear forces and inelastic rotations of the links showed much a smaller inelastic seismic demand for the eastern site, which can be attributed to inherent differences in seismicity and ground motion characteristics between eastern and western North America. This study also showed for that the amplification factors applied to link resistance to calculate design forces in the outer beam segments may be too conservative for eastern sites. Such a difference, already observed in past studies for EBFS and other structural systems, is not adequately accounted in current code provisions and should be investigated further for future code editions.

Two ground motion sets were assembled for nonlinear time-history analyses using different selection and scaling procedures. The comparison of the results revealed a similar force response of links and outer beam segments. However, the deformation response of these elements was very sensitive to the variability of ground motion records. Similar observations were made in previous studies which examined the seismic response of EBFS with shear-critical links. The results indicate that the EBF system, although highly ductile and effective in resisting seismic loads, can be subject to a highly varying deformation demand introduced by small variations in the ground motion input and may be prone to concentration of inelastic demand over the height. These characteristics must be taken into account when seismic design or seismic performance evaluation is based on the deformation response. The results also confirm that the selection and scaling of ground motion records must be considered with great care because they can have a significant impact on the assessment of structural damage which is usually expressed by deformation related parameters.

The analysis showed that the yielding of the outer beam segments did not occur frequently in the Montreal structures and the Vancouver 3-storey frame, and the inelastic rotational demand was limited. In the eight-storey Vancouver frame, the rotational demand imposed on the outer beam segments was more important at the top two storeys. Element fracture predictive tools were not implemented in OpenSees models and thus it was not possible to directly verify if the excessive inelastic deformations led to the local member failure. Thus, the peak inelastic rotations recorded in the outer beam segments were assessed using failure criteria that have been proposed in the literature. The inelastic beam rotation exceeded the rotational capacity limits only in the top storey for the ground motion set which induced more significant deformation response in all frames studied. Although the observed inelastic rotation may be considered excessive, it
was of short duration and the impact on global behavior is not known. It appears that for EBFs with long links, the yielding of the beams outside of the link is acceptable if the lateral stability is assured and the braces have sufficient stiffness and strength to resist part of the imposed end link moment. Such a design strategy could result in more economical designs which is particularly beneficial for EBFs with long links that yield predominantly in flexure. However, in view of the observed sensitivity of deformation response parameters to the characteristics of ground motion input and the high rotational demand recorded in outer beam segments for one of the studied structures, the general conclusions cannot be made. Large scale experimental studies investigating the global EBF response are necessary to determine the level of inelastic outer beam rotations that can be accepted without compromising the overall structural response.

ACKNOWLEDGMENTS

This study was financially supported by the Natural Science and Engineering Research Council of Canada through the Strategic Network Grant Program. The design of the studied structures was carried out by Mathew Laramée and Yacine Naciri, former graduate students at École Polytechnique de Montréal, and their contribution is gratefully acknowledged.

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