The impact of seismic response of outer beams, braces and columns on global seismic behaviour of chevron-type eccentrically braced frames

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SUMMARY:
The global seismic response of chevron-type eccentrically braced frames with shear-critical links is studied using nonlinear time history analysis. Yielding and flexural buckling of members other than links is tracked to examine its impact on overall frame performance. For frames designs compliant with capacity requirements, with a moment-resisting beam-to-brace connection, the rotational demand imposed on the outer beam segments was not excessive. Such demand can be easily accommodated, if the combined flexural strength of the braces and beams is not smaller than the expected end link moments and the braces can withstand the associated axial force-bending moment demand. The rotational demand imposed on the braces was also acceptable and did not adversely impact the global response. For designs not strictly compliant with capacity requirements, the study shows that structural failure is a possibility. The analyses also indicated that small differences in member sections can lead to significantly different deformation response.

Keywords: eccentrically braced frames, global seismic response, nonlinear time-history analysis

1. INTRODUCTION

Seismic design procedures for eccentrically braced frames (EBFs) are generally based on the premise that the inelastic activity is exclusive to links and aim to achieve elastic response of all other frame members, while maintaining link deformations bellow the acceptable limits. Previous studies (Richards and Uang 2006; Chao and Goel 2005; Koboevic 2000, Koboevic and Redwood 1997; Popov et al. 1992) indicated that elastic response of the frame elements other than links is not always achieved. Although it appears that the occurrence of limited yielding of outer beam segments should not jeopardize frame response as long as the lateral stability of the beam is provided (Engelhardt and Popov, 1989), inelastic behaviour of the outer beams is not integrated into current seismic design procedures in North America (AISC 2005, CSA 2005) in view of limited evidence regarding the possible extent of such yielding and its impact on overall frame response. Accepting the yielding in outer beam segments is a desirable design option because it avoids the possible strengthening of the beam needed to accommodate the high axial loads and bending moments introduced by yielding links.

Previous studies (Chao and Goel 2005, Koboevic 2000) also report that braces and the columns of the chevron-type EBFs may experience yielding, or even buckling during short periods of time. Although such a response is unlikely to be integrated in design, it is of interest to understand how this behaviour influences global frame response and under which circumstances, if any, it may be accepted.

In the study presented in this paper, 3- and 8-storey steel frame buildings with chevron-type shear-critical EBF bracing were designed for Vancouver with objective to examine global frame behaviour under seismic loads using the non-linear time history analysis. The attention is directed to the response of frame members other than links and the impact of their behaviour to the overall frame performance. The response of outer beam segments, braces and columns is first studied using ANSR-1 program (Mondkar and Powell, 1975) to determine the elastic seismic demand and detect the occurrence of the response not-anticipated in design process. More refined model is then built in OpenSees program (Mazzoni et al., 2006) to track and quantify inelastic response and study the impact on frame deformations. An alternative, 8-storey frame geometry, with longer, but still predominantly shear-critical links is also considered, to examine in more detail the influence of more substantial yielding of the outer beam segment on global response. Different strategies to design outer beams and braces are explored including the procedures fully compliant with current capacity design principles as well as non-compliant ones. Design recommendations are made in view of the observed response.

2. BUILDING DESIGN

Chevron-type EBFs with shear-critical links were designed for 3- and 8-storey buildings in Vancouver, BC (VCR3 and VCR8) assuming Class C site conditions (360m/s $\leq v_s \leq$ 760m/s). The typical floor plan adopted for both buildings is illustrated in Figure 2.1. Two EBFs studied provided seismic resistance in the north-south direction. The braced bay width was 8 m with 3.5 m and 4 m selected for the typical storey and the first storey respectively. Based on the results of the parametric study, a 600 mm length was chosen for links and it was decided to consider brace-to-beam connections as moment-resistant. All other connections were considered as pinned.

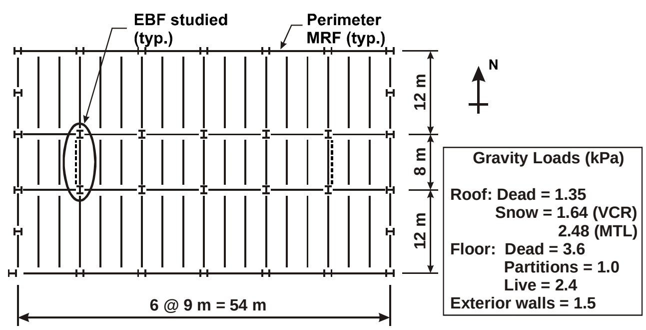

Figure 2.1. Design gravity loads and typical floor plan of the studied buildings

The structures were subjected to the gravity loads given in Figure 2.1. Wind loads were calculated as specified in NBCC 2005 (NRCC, 2005) but did not govern the design. The seismic base shear was determined using the static equivalent force method, employing the Equation 2.1:

$$V = \frac{S(T_a) M_v I_E W}{R_d R_o} \tag{2.1}$$

where T_a = empirical structural period ($T_a = 0.025h_n$, h_n being a total height of the structure); $S(T_a)$ = spectral acceleration at design period based on probability of exceedance of 2 percent in 50 years and modified by site coefficients F_a and F_v to reflect the soil conditions; M_v = factor reflecting the increase in base shear due to higher mode effects; I_E = structure importance factor; W = total seismic weight tributary to the frame; R_d = ductility factor and R_o = overstrength factor. In this study, R_d = 4.0; R_o = 1.5; $F_a = F_v$ = 1.0; I_E = 1.0, and M_v = 1.0. The force V from Equation 2.1. needs not exceed 2/3 of the value determined at T_a = 0.2 s but must not be less than the value computed at T_a = 2.0 s. The design base shear was determined using the increased period equal to 2.0 T_a, as permitted by NBCC 2005. This assumption was justified in the subsequent modal analysis. Accidental torsion was not considered to ensure the compatibility with 2D non-linear analysis. The calculations are summarized in Table 2.1.

Table 2.1. Fundamental period and seismic design base shear for studied structures

Structure	Design period $T=2T_a$ (s)	Base shear (V) (kN)	S(T) (g)	V/W (%)
VCR3	0.55	2029	0.609	10.15
VCR8	1.43	2658	0.262	4.37

The design was conducted in compliance with CAN/CSA S16-01 (CSA 2005) seismic provisions. The links were selected to have the adequate inelastic shear resistance for factored seismic loads including P-Delta effects. The braces and the outer beams were designed as beam-columns to resist the forces introduced by $1.3R_y$ times the nominal shear resistance of the links, V_p. Beams were selected from W shapes and the braces from an HSS section database. The beams were considered as fully laterally supported by the floor slabs. The combination of bending moments and tensile axial loads was critical for these elements. Brace design was governed by the combination of compressive axial loads and bending moments. At link ends, 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 as yielding of the outer beam segment was deemed acceptable in design.

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 $1.15R_yV_p$ for the top and lower tiers respectively). The bending moments resulting from column continuity and the relative storey movements were considered as required by CSA-S16.

The structures were then verified for adequate stiffness and the strength under all relevant load combinations including gravity loads, notional loads, wind and seismic loads. These verifications lead to a small increase in size of top storey columns and braces of three-storey frames only. The inter-storey drift requirements of NBCC did not control the design of any of the frames studied.

3. STUDY OF STRUCTURAL RESPONSE

3.1. Study of behaviour using ANSR-1 program

Initial study of the seismic response of the frames was done using ANSR-1 program. The macro-element developed by Ricles and Popov (1994) was selected to model the link behaviour. The element is composed of an elastic beam with end plastic hinges that are divided into three sub-hinges with inelastic behaviour both in shear and in flexure. Properties of the link elements were determined by calibration against data from 11 cyclic tests on short shear links by Okazaki et al. (2005).

Elements other than links were modeled using beam-column element that combines the elastic beam, concentrated end plastic hinges with axial force-bending moment interaction, and rigid end zones. The cross-section yielding of the laterally supported outer beam segments under combined bending moment and axial force could be adequately represented. The beam strength was calculated using expected yield strength, R_yF_y, where $R_y = 1.1$. However, this element cannot be used to represent the possible flexural and/or lateral-torsional buckling failure modes that can occur for braces and columns. Hence, for these frame members, the elastic behaviour was described.

Nonlinear time history analysis was carried out for two sets of ground motion records. Ten historical records were selected from the PEER database (PEER 2006) based on dominant magnitude-distance scenarios and local site conditions. In addition, 14 synthetic records from the ground motion database presented in Tremblay & Atkinson (2001) were considered. The calibration was done by matching the spectral intensities of the record and NBCC 2005 design spectrum for Vancouver over the range of periods determined on basis of the best visual fit between the two spectra. More details can be found in Rozon et al. (2008).

For a given record, the elastic demand on the braces and columns was determined at every time step, by examining the bending moment-axial force interaction. The total time during which any of the columns or braces showed yielding or buckling was determined and summed up for all ground motions and for all braces and columns of one frame. The verification of interaction equations was done using the member expected resistance but also the factored resistances considered in the design.

For the outer beam segments, for which the cross-sectional yielding was modelled, inelastic rotation excursions were tracked directly and the extent of yielding was determined using the same approach. The results are summarised in Table 3.1. Note that the median values of shear forces in the links slightly exceeded those anticipated in design reaching the maximum of $1.41V_p$ and $1.47V_p$ for 3-storey and 8-storey frame respectively. The mean values of the accompanying inelastic shear rotations of the links in 8-storey frame also exceeded the design values reaching the maximum of 0.093 rad.

Table 3.1. Number of records and total duration of inelastic excursions in frame members other than links

	3-storey frame				
	Outer beams	Braces		Columns	
Resistance	Expected	Factored	Expected	Factored	Expected
Number of records (/24)	13	22	6	15	1
Duration (s)	1.35	30	1.1	6.3	0.05
	8-storey frame				
Number of records (/24)	3	24	5	19	3
Duration (s)	0.33	60.8	0.52	17.5	0.21

As anticipated in design, the outer beam segments in both frames did experience yielding, although generally very small. This can be attributed to the relatively small portion of link end moments transferred to the outer beams as a result of the selected frame sections, geometrical configuration and link lengths. Excessive elastic force demand on braces and columns was more noticeable in the 8-storey frame. Flexural buckling was predicted during 61 s for the braces in the upper and bottom storeys when the factored resistance was considered in verifications. The portion of the link end moment that was transferred to the braces (M_{brace}/M_{link}), was approximately 10% higher than the value considered in design which can explain brace overload. Flexural buckling was also predicted in the bottom-storey columns, where the axial forces and bending moments slightly exceeded the design predictions. This difference can be attributed to the higher levels of strain-hardening that developed in links. When the expected resistance is used in the verifications, the response is predominantly elastic and the occurrence of flexural buckling significantly reduces both for braces and columns.

3.2. Study of behaviour using OpenSees program

In order to include the inelastic response of braces and columns, and further investigate the impact of such behaviour on overall frame response, a refined analytical model was built in the OpenSees program. Typical features of the model are illustrated in Figure 3.1 for a 3-storey frame.

Modelling of the links follows the recommendations by Ramadan and Ghobarah (1995) and Richards and Uang (2006) who reinterpreted theoretical concepts developed by Ricles and Popov (1994). The link macro-element is replaced by an elastic beam and a series of zero length rotational and translational springs exhibiting bilinear force-deformation response that are combined to represent the yielding response of the whole element. Similarly to ANSR-1 model, the properties of the link elements in OpenSees were determined by calibration against experimental data (Okazaki et al., 2005).

The outer beam segments, braces and columns were modeled using eight nonlinear beam-column elements with fibre discretization of the cross section to reproduce cross-sectional yielding as well as in- and out-of plane flexural buckling. Initial member out-of-straightness was specified for these elements. Each element included 4 integration points and a total of 16 fibres were used to model the cross-section, as recommended by Aguerro et al. (2006). 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 as modified by Lamarche and Tremblay (2008) to account for residual stresses. The residual stress pattern proposed by Galambos and Ketter (1959) was used in the analysis. Note that the columns were orientated in such a way that the bending moment arising from column continuity and the relative storey movements were induced about the strong column axis. In this case, the column failure could be governed by lateral-torsional buckling and this condition should be represented in the model. For the frames studied however, that failure mode was not critical and therefore the modelling of cross-section yielding and flexural buckling was judged sufficient.

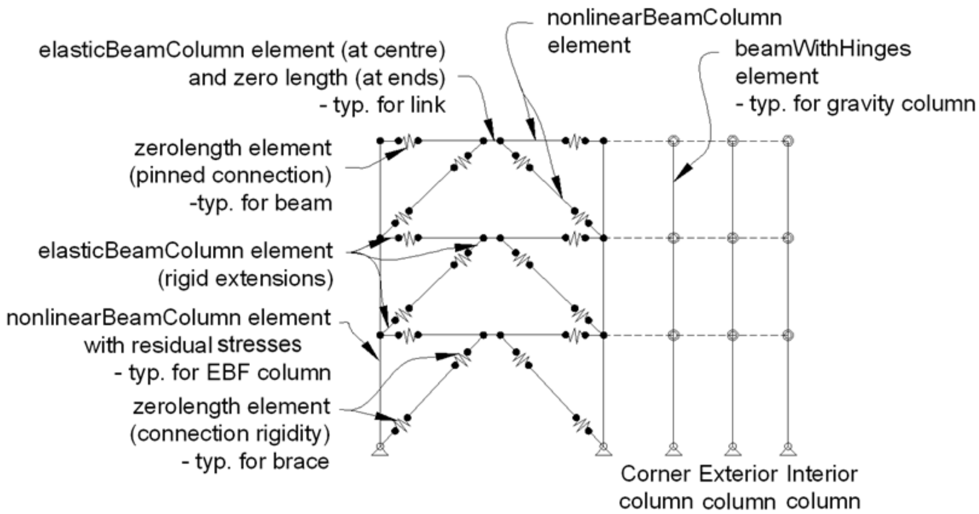

Figure 3.2. OpenSees model of three-storey EBF

Three series of analyses were performed with different levels of resistance considered for the elements other than the links: (i) infinite resistance to obtain a reference elastic response; (ii) expected resistances, using R_yF_y = 385 MPa; and (iii) factored resistances using R_yF_y = 315 MPa. The response of links was first compared to that obtained by ANSR-1 program. The two models gave almost identical results for the median values of maximum shear forces and deformations. Sensitivity of deformation parameters to an individual earthquake record used in the analysis was observed when comparing 84th percentile results. Some yielding and instability of the members other than links was recorded, but no indication of possible collapse due to member or global instability was detected in any of the analyses.

In order to examine a more critical case and introduce a greater flexural demand on the beam segments outside of the links, the geometry of the 8-storey frame was modified by increasing the length of the links from 600 mm to 800 mm. No modification to dimensions of link beams or columns were required in consequence, but some lengthened links now fell into intermediate-length category with $1.1 \leq eV_p/M_p \leq 2.1$. However, new brace profiles had to be selected. Three approaches to brace design were considered, each of which lead to different level of conservatism in beam design: (a) Moment-resistant connection is assumed between the braces and the beams, and the braces are selected to attract a portion of the link end moment large enough to ensure fully elastic response of the outer beam segments. Link end moments, are distributed between the braces and the outer beams in proportion to their respective flexural rigidity. (b) Moment-resistant connection is assumed between the braces and the beams, but inelastic response of the outer beam segments is accepted. The rotation compatibility between the beams and the braces at the brace-to-beam connections thus need not be satisfied. The braces are selected such that the combined flexural strength of the brace and the beam outside the link, in presence of the concomitant axial loads, exceeds the link end moment. This design strategy essentially corresponds to the one that was adopted in initial design. Once the braces are selected, the flexural rigidities of the braces and beams are recalculated to verify that the initially assumed bending moment distribution still applies. (c) Pinned connection is assumed between the braces and the beams, thus implying that the entire link end moment is transferred to the outer beams. Consequently, only the axial load is considered in brace design. In that case, however, the outer beam is not verified for that flexural demand, implying that significant yielding might develop in that beam segment. In the model, the brace-to-beam connection is considered as moment-resistant because these connections typically exhibit flexural stiffness and strength, even though they were designed as pins. Hence, while the braces conform to capacity design approach in terms of axial loads, both the braces and outer beam segments are undersized for flexure, which permits to evaluate the impact of more pronounced yielding or flexural buckling of these members on global frame response.

In all three cases, the end link bending moments were calculated assuming the link shear forces will reach $1.3R_yV_p$, the axial and flexural resistances of the beam segments outside of the links were taken equal to factored resistances amplified by the R_y/ϕ ratio, and factored resistances were used for the braces.

The steel tonnages required for all braces of the frame for the three designs were, respectively, 6190, 5800, and 4120 kg. Braces selected in design (c) were about 70% lighter compared to designs (a) and (b). The critical demand-to-capacity ratios for the outer beam segments are given in Table 3.2.

Table 3.2. Demand-to-capacity (D/C) ratios for the outer beams of the 8-storey frame (modified designs)

Storey	Case (a)	Case (b)	Case (c)	Case (d)
	D/C			
8	0.97	1.0	1.56	0.85
7	0.93	1.0	1.40	0.87
6	0.98	1.0	1.23	0.85
5	0.97	1.0	1.15	0.83
4	0.97	1.0	1.17	0.84
3	0.97	1.0	1.17	0.84
2	0.97	1.0	1.25	0.87
1	0.92	1.0	1.04	0.84

To assess the potential for local member fracture, in Table 3.3, the inelastic rotational demand imposed on the outer beam segments for three brace designs is compared to the rotational capacities calculated using the approaches proposed by Kemp (1996) and Okazaki (2006). The rotational demand and the rotational capacity are normalised with respect to the elastic rotation that develops when M_p is reached.

The experimental data used in Kemp's formulation were obtained for monotonic loading, and therefore the values may be unconservative for the earthquake loading. The original formulation by Okazaki et al. is based on cyclic test data but does not account for the presence of axial load. The rotational capacity values obtained using the original Okazaki's formulation were thus modified by the factor $\alpha = 2h_c/h$ proposed by Kemp to obtain more realistic limits on beam rotational capacity. In this expression, h_c is plastic depth of web in compression for the beam and h is the total beam depth. Note that, even though Okazaki's tests were carried out for cyclic loading, the study targeted beams in moment-resisting frames and thus the calculated rotational capacities given above may be somewhat severe to judge the inelastic performance of the outer beam segments in eccentrically braced frames.

Table 3.3. Rotational demand and capacities for the outer beam segments of the modified 8-storey frame

Storey	Rotational demand $(\theta-\theta_e)/\theta_e$									Rotational capacity	
	Case (a)			Case (b)			Case (c)				
	50^e	84^e	max	50^e	84^e	Max	50^e	84^e	max	Kemp	Okazaki
8	0.00	0.00	0.00	0.00	0.00	0.01	0.22	0.44	0.65	1.41	0.64
7	0.00	0.00	0.00	0.00	0.00	0.00	0.34	0.48	0.58	1.80	1.03
6	0.00	0.01	0.01	0.01	0.01	0.02	0.13	0.20	0.39	1.70	0.51
5	0.00	0.01	0.01	0.01	0.01	0.02	0.02	0.10	0.24	1.77	0.56
4	0.00	0.01	0.01	0.01	0.01	0.04	0.05	0.09	0.17	2.00	1.03
3	0.00	0.01	0.01	0.01	0.01	0.02	0.02	0.06	0.26	1.79	1.07
2	0.01	0.02	0.26	0.04	0.10	0.51	0.18	0.65	2.63	2.98	2.16
1	0.01	0.02	0.06	0.01	0.02	0.06	0.02	0.05	0.57	1.41	0.64

The rotational capacities determined according to Kemp`s formulation vary between 1.41 and 2.98 while the rotational capacities obtained by Okazaki's method are significantly smaller, varying between 0.56 and 1.07 in all but the first-storey beam for which 2.16 was reached. The median and 84th percentile beam rotational demand for the three design cases studied is below the Okazaki limits. Almost no yielding was observed for design (a), which is consistent with the targeted elastic response of the outer beam segments. In design (b) some inelastic activity was observed, and thus higher rotations were recorded, particularly in the second-storey beam. As expected, the largest demand was observed for design (c) with maximum median and 84th percentile values reaching 0.34 and 0.65 respectively. Note that for this design case, the rotational demand in one ground motion record (2.63) exceeded the Okazaki limit at the second storey.

Table 3.4. presents the brace plastic rotational demand obtained for the three designs. Brace plastic rotation follow the occurrence of brace flexural buckling in compression. The amplitude of this rotation is in function of the maximum negative axial deformation (axial deformation in compression) reached in the analysis and the maximum plastic elongation experienced by the brace prior to that point. Rotational capacities shown in Table 3.4. were obtained from the empirical relationship proposed by Tremblay et al. (2003) based on cyclic tests on HSS braces. The maximum rotation recorded in the third storey brace was less than 54% of its rotation capacity. Demand gradually increases from design (a) to (c), but in all cases, it remained well below the brace fracture capacities.

Table 3.4. Plastic rotational demand and capacities for the braces (in radians) of the modified 8-storey frame

Storey	Case (a)			Case (b)			Case (c)		
	50^{e}	max	limit	50^{e}	Max	limit	50^{e}	max	limit
8	0.032	0.035	0.153	0.031	0.034	0.172	0.038	0.044	0.177
7	0.036	0.038	0.136	0.034	0.035	0.166	0.046	0.049	0.164
6	0.039	0.040	0.146	0.036	0.037	0.165	0.044	0.046	0.178
5	0.040	0.041	0.148	0.040	0.041	0.165	0.050	0.054	0.160
4	0.038	0.039	0.159	0.043	0.044	0.148	0.049	0.052	0.166
3	0.042	0.047	0.159	0.053	0.064	0.148	0.062	0.079	0.146
2	0.037	0.039	0.145	0.041	0.044	0.145	0.053	0.078	0.146
1	0.043	0.045	0.162	0.043	0.046	0.162	0.050	0.060	0.151

All three designs demonstrated adequate seismic response at median and 84th percentile level. The rotational demand imposed on the outer beam segment in designs (a) and (b) can be easily accommodated, confirming that yielding of these beam segments could be accepted in design. When the outer beams are not designed in compliance with capacity design requirements, such as in design (c), ground motion records could impose rotational demand exceeding the member inelastic capacities which, in turn, may adversely impact frame behaviour.

To evaluate further the extent to which capacity design requirements could be relaxed without compromising EBF response, a fourth design case (d) was examined. This design approach was identical to design (a) except that the factor 1.3 was reduced to 1.0 when calculating the expected link resistance to determine design forces for the outer beams and braces. Beams sections remained unchanged and, coincidentally, the resulting brace sections were very similar to those selected in design (c). Nonlinear dynamic analysis showed acceptable rotational demand at median level, but two bottom beams developed inelastic rotations that exceeded both Okazaki and Kemp limits by a large margin at the 84th percentile level. The maximum rotational demand observed in the second–storey beam (11.7) was 6.5 times the Kemp's limit. Similar high plastic demand-to-capacity ratios were observed for the braces. The rotational capacity was inadequate for the bottom two braces, both at 84th percentile and the maximum level. At the first storey, fracture of the brace was predicted in six out of the 24 records studied and the maximum rotation reached 3 times the limit. For that design, inter-storey drifts reached up to 3.88% h_s at the 84th percentile level and exceeded 5% h_s in four of the records, the collapse limit suggested by Tremblay and Robert (2001) for concentrically braced frames. These results indicate that this design approach is unsafe and may lead to structural failure. The close resemblance between the bracing members in cases (c) and (d) and the observed sensitivity of the EBF systems to inelastic deformations, suggest that design approach (c) should not be recommended before further study is completed.

4. CONCLUSIONS

The global seismic response of 3- and 8-storey eccentrically braced steel frames designed for typical western North American location (Vancouver) was investigated for selected ground motions representative of 2% in 50 year hazard level. The attention was directed to the response of frame elements other than links with objective to validate if the current design procedure achive desired elastic and stable behaviour of these elements, and to investigate the impact of possible non-desired response on the overall frame seismic performance. The reported results were obtained for chevron-type EBFs with shear-critical links and are representative of the behaviour of the bare steel frame. In a real structure such situation occurs when the slab is non-composite and/or is detached from the steel frame in the link zone.

OpenSees model permitted a more refined representation of the behaviour of frame members other than links including cross-section yielding of the outer beam segments, as well as cross-sectional yielding and flexural buckling of braces and columns. The impact of such behaviour on overall seismic performance of the frames could therefore be assessed. For designs compliant with capacity requirements, with moment-resistant brace-to-beam connection and the brace designed as a beam-column, the rotational demand in the outer beam segments was not excessive and can be easily accommodated. This confirms 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. Such a design strategy could result in more economical design. Additional experimental and analytical data is however required to verify if it could be extended to EBFs with flexural links and other geometric configurations.

The study confirmed that braces and the columns of the EBFs designed in accordance with current seismic design provisions may experience cross-sectional yielding and flexural buckling. When proper capacity principles were applied in design, inelastic rotational demand on the braces was acceptable and did not have any negative impact on the global frame response. Conversely, when capacity design requirements were not strictly respected, the study showed that structural failure can take place. The sensibility of frame deformation response to small variation in member sections (brace sections in this study) and ground motion input was also detected in this study.

Lateral-torsional buckling of columns bent about their strong axis was not critical for selected EBF configuration and, that failure mode was therefore not examined. However, this mode may govern the failure of the columns in EBFs with different geometric configurations. The impact of such behaviour on global structural response will be addressed in future studies.

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