SEISMIC PERFORMANCE OF ECCENTRICALLY BRACED FRAMES DESIGNED FOR CANADIAN CONDITIONS

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ABSTRACT:

This paper describes an initial phase of a comprehensive study investigating seismic behaviour of chevron-type eccentrically braced frames (EBFs) designed according to Canadian design requirements. Attention is directed to the taller frames with shear-critical links, located in typical eastern and western North-American sites. Design procedures are described and applied to the 14- and 20-storey frames. The importance of different design criteria is discussed and the appropriate design sequence is suggested. It was found that ductility requirements did not control design. The seismic response of the frames was investigated using the non-linear time-history analysis to assess if the design procedures achieved desired frame response. The analyses were done for the sets of earthquake records calibrated to match design spectra at studied locations. The results obtained indicate that structures in Vancouver developed links shear forces and deformations higher than those anticipated in design. In spite of large reserve of strength, limited yielding of braces and columns was also observed. For structures in Montreal all global and local response indicators remained well bellow design limits.

KEYWORDS: steel structures, eccentrically braced frames, design norms, non-linear analysis

1. INTRODUCTION

Eccentrically braced steel frames (EBFs) dissipate energy induced by earthquake loading through the inelastic deformations of beam segments called links. These segments should be preferably short and centrally placed to promote the shear yielding and avoid possible problems related to the connections between the beam and the column. Design procedures are based on capacity design principles and aim to produce frames with stable inelastic response of links and elastic behaviour of all other frame members. To achieve this behaviour, links are selected to have adequate inelastic resistance for factored seismic loads while columns, braces and outer beam segments are sized for the forces generated by fully-yielded and strain-hardened links. Verification of link inelastic rotations completes the ductility phase of design. Selected sections are also checked for the ultimate and serviceability limit states for all relevant load combinations, including wind and earthquake.

Previous studies investigated mainly the response of lower to medium height EBFs designed for western North-American locations. It was shown that, for the regions with higher seismicity, it was possible to obtain efficient designs. In general, frame members were highly utilised while providing adequate structural strength, stiffness and ductility. Non-linear time-history analyses suggested however that desired frame behaviour was not always obtained; links in the upper storeys developed higher shear forces and deformations than anticipated in design, columns, braces and outer beam segments showed inelastic behaviour and inelastic inter-storey drifts remained well bellow the values predicted in design. The present study focuses on the seismic response of taller frames. In such structures larger deflections are anticipated and thus providing the adequate stiffness and global structural stability could become important design consideration. In zones with lower seismic activity design could be governed by wind or gravity loads. Consequently, it is expected that taller EBFs will be differently proportioned compared to the lower EBFs which might significantly alter their seismic behaviour.

The present study is conducted with the objective to investigate the particularities of design and seismic response of taller EBFs. The impact of different design phases is evaluated by monitoring the increase in structural mass, so that the most appropriate design sequence for different frame heights and design locations can be suggested. Seismic response of these frames is studied using non-linear time-history analysis and design requirement are critically reviewed in light of the results obtained. The EBFs selected for this study are all chevron-type with short shear links. Frames were designed for two Canadian locations representative of eastern and western North-American seismic conditions. Three different heights (14, 20 and 25 storeys) were considered. Results and discussions presented herein are those obtained for 14 and 20-storey frames.

2. DESIGN

2.1 Outline of design procedure

Canadian requirements for seismic design of EBFs are provided in the National Building Code of Canada (NBCC 2005) and the steel design standard (CAN/CSA-S16-05). The design process is iterative, and initial member sizes are usually determined to provide adequate ductility. For EBFs with shear links, link sections with adequate inelastic shear resistance for factored seismic loads are first selected. Applying capacity design principles, forces used to design the other frame members are calculated using the amplified expected resistance of the link[1] to ensure that the links remain the weakest elements in the frame. Link inelastic shear rotations are calculated and compared with design limits. The initial design is then checked for strength and stiffness requirements for all relevant load combinations including P-Δ effects, and the member sizes are modified if required. For newly selected sections, design sequence is repeated until the satisfactory design is obtained.

To minimize the number of iterations and in view of the importance of deflections in design of taller structure, the design sequence was modified in this study. The following steps were adopted: (i) select links for factored seismic loads; (ii) select other members based on the strength demand for all relevant load combinations including gravity loads, notional loads, wind and seismic loads; (iii) verify inelastic-drift requirements; (iv) calculate global stability factor, U_2, and compare to suggested values[2]; (v) adjust design forces to account for P-Δ effects; (vi) verify strength, stiffness and stability for increased forces; (vii) conduct verification of beams, columns and braces for forces induced by links (capacity design) and (viii) verify link inelastic rotations.

2.2 Design of 14 and 20-storey frames

14- and 20-storey frames were designed for two Canadian locations, Montreal, QC, and Vancouver, BC, assuming Class C soil. The layout and the elevation of 14-storey frame are shown in Fig. 2.1.

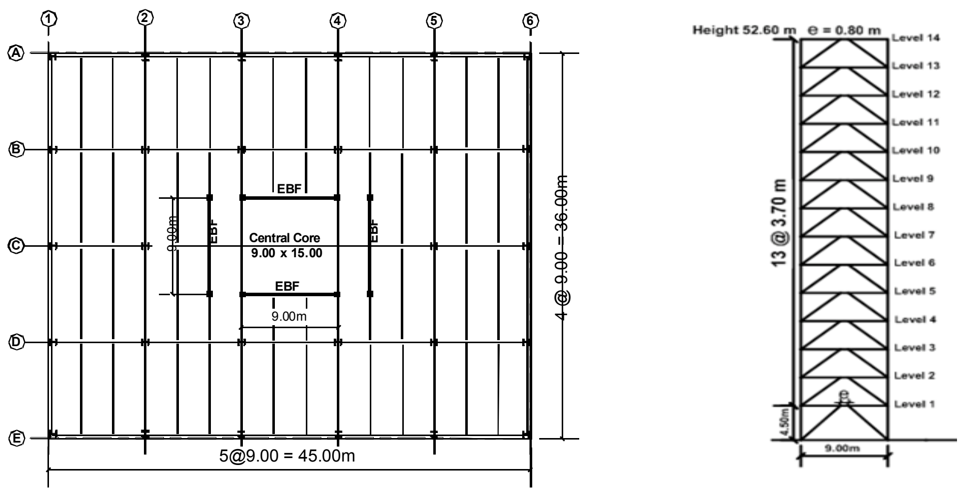

Fig.2.1. Layout and the elevation of the fourteen-storey EBF

[1] For outer beams and braces : $1.3R_yV_p$, where R_y is the ratio between expected and nominal yield stress ($R_y = 1.1$)
For columns: $1.3R_yV_p$ for top tier and 1.15 R_yV_p elsewhere
[2] NBCC 2005 suggests maintaining the loads increase bellow 40% (i.e. $U_2 \leq 1.4$) to prevent potential instability during strong earthquake.

Two symmetrically placed EBFs brace the building in each orthogonal direction. The braced bay width was 9 m and links were 800 mm long. This length was chosen after the parametric study showed that it would be very difficult to maintain inelastic shear rotations below design limits if the shorter links (600 mm) were used. Typical storey heights were 3.7 m with 4.5 m at the first storey. For 20-storey frame the same layout was retained and six typical storeys were added resulting in the total frame height of 75 m.

The structures were subjected to gravity loads previously used by Koboevic (2000). Wind loads were calculated as specified in NBCC 2005. The base shear was determined following the static equivalent force method. The augmented empirical period ($2T_a$) was used as permitted by the Code to account for the fact that the empirical estimates of the periods are conservative. Subsequent modal analyses showed that this assumption was justified. It was found that for all cases studied, the condition for the minimum base shear defined in Eqn. 2.1 governed.

$$V = \frac{S(T_a)M_v I_E W}{R_d R_o}\,;\; V \geq \frac{S(2.0)M_v I_E W}{R_d R_o}\,;V \leq \frac{2}{3}\frac{S(0.2)M_v I_E W}{R_d R_o} \qquad (2.1)$$

In this equation $T_a = 0.025h_n$, where h_n is a total height of the structure; $S(T_a)$ is the spectral acceleration at design period based on probability of exceedance of 2 percent in 50 years and modified by foundation coefficients F_a and F_v to reflect the soil conditions; M_v is the factor accounting for the increase in base shear due to higher mode effect; I_E is structure importance factor; W is total seismic weight tributary to the frame; R_d is the ductility factor and R_o is the overstrength factor. In this study, $R_d = 4.0$; $R_o = 1.5$; $F_a = F_v = 1.0$ and $I_E = 1.0$. Summary of design base shear calculations are given in Table 2.1.

Table 2.1 Summary of design base shear calculations

Number of storeys and design location	14 STOREY EBF				20 STOREY EBF			
	Vancouver (VCR14)		Montreal (MTL14)		Vancouver (VCR20)		Montreal (MTL20)	
Seismic weight, *W* (kN)	104915		105255		152249		152589	
Design period, $2T_a$ (s)	2.63		2.63		2.63		2.63	
Spectral accelerations at 0.2s and 2s S(*0.2*); S(*2*) (g)	0.94	0.17	0.69	0.048	0.94	0.17	0.69	0.048
M_v	1.0		1.115		1.0		1.348	
Design base shear (kN)	17836		7578		25882		10986	

The following features were common for all designs: columns have pinned connection at the base, they are continuous over the height and tiered into two-storey segments, brace-to-beam connections are moment-resistant thus re-distribution of the link end moment is possible, and beam-to-column and brace-to-column connections are pinned.

In general, for all four frames, gravity and wind load combinations required stronger links compared to ones needed for seismic loads. This difference was more pronounced for frames in Montreal. Even greater overstrength was observed for the top storey links where stronger sections had to be selected in order to have shear-critical link or Class 1 section. For typical storeys, the ratio between link inelastic shear resistance and link force demand, α, was about 1.3 for Vancouver and 1.8 for Montreal. In top storeys the average value of a was about 2.5 for all frames with the exception of 14-strorey frame in Montreal where the top link was 4.5 times stronger than required for seismic loads.

Several subsequent design steps were carried out using the computer program Visual Design. This permitted automatic section sizing in compliance with strength design requirements for governing load combinations. Automatic features were not applied neither to verify stiffness nor to include second-order effects since these checks required the use of inelastic displacement estimates, a feature not available in the program. At the end of each design phase, the mass of the structure was calculated to determine which criteria were critical for design. Results obtained after strength, stiffness and stability verifications are presented in Table 2.2.

Table 2.2 Structural mass in different phases of design

Design phase	Structural mass (kg)			
	14-storey frame		20-storey frame	
	VCR14	MTL14	VCR20	MTL20
Strength	34367	31974	83649	70785
Stiffeness ($\Delta \leq 0.025h_s$)	50915	36228	191693	106456
Stability ($U_2 \leq 1.4$)	64056	63511	237246	214600

Note that minor changes of section sizes were required in the final ductility verification thereby the structural mass obtained after modifications for stability requirements is practically the same as the mass of the final design. As can be seen from Table 2.2, in spite of an important difference in seismic solicitations in two locations studied, the mass obtained for frames with equal height was virtually the same. Comparison of structural mass obtained in different steps of design process indicate that for the frames in Vancouver, inelastic-drift requirements governed the design, while for the frames in Montreal global structural stability was the main criterion. A closer examination of the strength design results shows that, with the exception of top storey members, for all four frames wind load combinations were critical. Final designs obtained for 14-storey frames are summarized in Table 2.3.

Table 2.3 14-storey frame: Summary of selected shapes (steel CSA-G40.21-350W)

	Frame VCR14 (Mass = 64060 kg, T = 2.86s)			Frame MTL14 (Mass = 63550 kg, T = 2.98 s)		
Storey	Braces	Columns	Beams	Braces	Columns	Beams
14	HSS254x254x9.5	W 200x59	W 200x42	HSS203x203x11	W 360x79	W200x42
13	HSS305x305x13		W200x52	HSS305x305x11		W200x42
12	HSS305x305x13	W 360x134	W250x58	HSS305x305x11	W 310x118	W250x45
11	HSS305x305x13		W310x67	HSS305x305x13		W250x58
10	HSS305x305x13	W 460x235	W360x72	HSS305x305x13	W 460x193	W250x67
9	HSS305x305x13		W360x79	HSS305x305x13		W360x64
8	HSS305x305x13	WWF 450x308	W410x74	HSS305x305x13	WWF 500x381	W360x72
7	HSS305x305x13		W460x86	HSS305x305x13		W360x79
6	HSS305x305x13	WWF 500x456	W410x85	HSS305x305x13	WWF 550x503	W410x67
5	HSS305x305x13		W460x89	HSS305x305x13		W410x74
4	HSS305x305x13	WWF 600x551	W530x74	HSS305x305x13	WWF 600x680	W460x68
3	HSS305x305x13		W530x85	HSS305x305x13		W460x82
2	HSS305x305x13	WWF650x739	W530x101	HSS305x305x13	WWF650x598	W460x82
1	HSS305x305x13		W610x113	HSS305x305x13		W530x101

NBCC 2005 stipulates that the equivalent static load method could be used to represent dynamic response of tall buildings only if the structures are regular, less than 60 m high with fundamental periods below 2 sec. In all other cases, it is assumed that the dynamic response becomes significantly influenced by the higher modes so that simplified assumptions used in the equivalent static method may no longer be valid. To investigate the impact of seismic load distribution on final designs, all four structures were re-designed using lateral load profiles obtained from modal analysis.

Fig. 2.2 shows the load distribution for the 14-storey structure in Vancouver. The load profile obtained applying the equivalent static load method is also plotted. Sensitivity of the results to the variation in member sizes is evaluated by conducting the modal analysis for two distinct structures; Frame A in which the sections were selected respecting only the strength requirements for factored loads combinations, and Frame B having the same beams as Frame A, but with all braces and columns identical to those of the first storey in Frame B. As can be seen from Fig. 2.1, the obtained load profiles for the two cases were very similar. However they differ significantly from the distribution predicted by the equivalent static load approach, clearly indicating the importance of the higher modes. Had the design been controlled by seismic loads, either regarding the required strength or ductility, the resulting frame designs would have been different showing perhaps different inelastic response to seismic solicitations. It was demonstrated earlier in the text that for the selected frames other requirements were critical for design. For this reason, final frame designs obtained using modal and equivalent static load distributions showed very little difference. For the reasons of simplicity it was therefore decided to proceed to non-linear time history analysis with designs obtained using seismic load profiles obtained from the equivalent static force method.

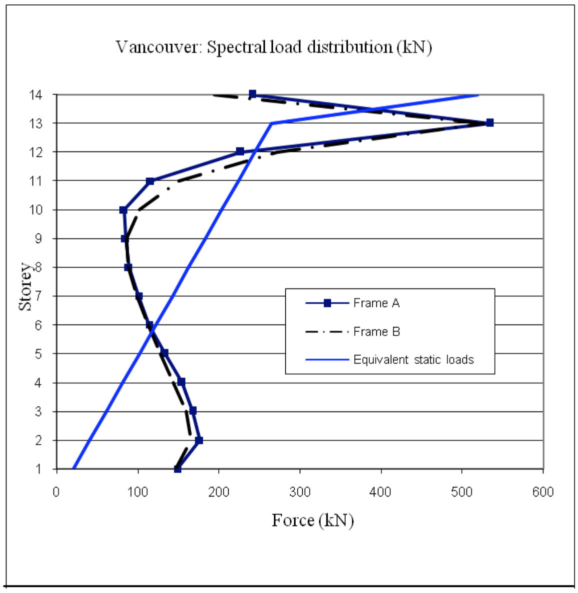

Fig. 2.2 Lateral force distributions obtained from the spectral analysis

3. NON-LINEAR DYNAMIC ANALYSIS

3.1 Modelling for the analysis and selection of earthquake records

Non-linear time history analyses were conducted using computer program ANSR-1 with special link element developed by Ricles and Popov (1994). The element consists of an elastic beam with plastic hinges concentrated at its ends that can yield both in shear and in flexure. Both isotropic and kinematic strain-hardening are represented. Each hinge is divided into three sub-hinges that have inelastic behaviour both in shear and in flexure. The interaction between moment and shear in inelastic range is not considered, and the axial deformations are neglected. These simplifications are acceptable in view of the experimental evidences on inelastic behaviour of short shear links. The link element was calibrated[3] using the results of experimental studies conducted by Okazaki et al. (2005).

Other frame members are modelled using standard beam-columns elements. It was assumed in the design that limited yielding of outer beam segment is acceptable as long as the brace and the outer beam segment together can sustain the total link end moment in combination with the axial forces introduced by yielded and strain-hardened links. These elements were thus represented by inelastic beam-columns elements for which the cross-section yielding under combined bending moment and axial force was described. Braces and columns were modelled using elastic beam-column elements and their response was subsequently examined by tracing the time-history of bending moment-axial force interaction.

[3] Rozon, J. 2008. Personal communication

Non-proportional damping was specified where mass-proportional viscous damping was assigned to the links and Rayleigh damping based on 3% of critical, was assigned to the other frame members. Masses were concentrated on column lines and P-Δ effects were represented by fictitious pin-connected columns carrying the part of the tributary seismic weight not assigned directly to the frame columns.

For each design location an ensemble of acceleration records was defined and scaled so that the compatibility with design spectra is achieved. For Vancouver site 10 historical and 10 artificial [4] records were selected while the Montreal suite comprised of 10 artificial accelerograms. For each accelerogram the scaling was done using the hybrid method[3] in which the intensities of the record spectrum and design spectrum were matched over the range of periods determined on basis of the best visual fit between the two spectra.

3.2 Response of the 14-storey frame

The results presented herein pay particular attention to the maximum induced link shear forces and deformations, the characteristics of the response of other frame members and the inelastic inter-storey drifts. For each accelerogram maximum values of response parameters were found at every storey and median and 84th percentile values were calculated.

Fig. 3.1 illustrates results obtained for maximum link shear forces and deformations for both frames. Link forces are normalized by the probable shear resistance (V_pR_y). Both for Vancouver and Montreal median and 84th percentile values obtained show similar trend. Very little yielding was observed in the links located in the middle portion of the frames. Maximum values of strain-hardening were attained in the upper storeys. In Montreal, normalized link forces stayed below the limit anticipated by Canadian steel design standard (1.3), while in Vancouver the larges median value of 1.5 was reached. This magnitude of strain-hardening is consistent with the experimental observations reported by Okazaki et al. (2005).

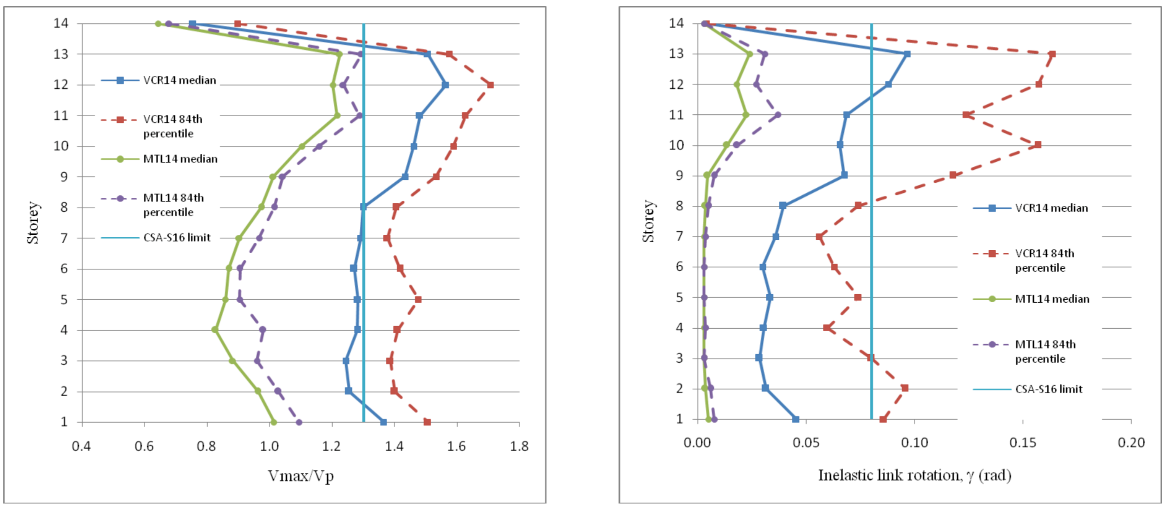

Figure 3.1 14-storey frame: Normalized maximum link shear forces and deformations

The same figure shows the results obtained for the inelastic shear link rotations, γ. The design procedure restricted the value of γ to 0.08 rad. Significant differences in response can be observed for two location studied. Ductility demand imposed on the links in MTL14 structures was small, and concentrated in the upper storeys. Even the 84th percentile values stayed well below the design limit reaching the maximum value of 0.025 rad. In Vancouver, higher values of γ were obtained, all in the upper portion of the frame. In the 12th and 13th storey, median values exceeded the design limit, by a maximum of 20 percent. 84th percentile values are much higher (max. 0.15 rad) and exceed significantly the design limit.

[4] Atkinson, G. 2007. Personal communication

Outer beam segments of structure MTL14 responded elastically, while some yielding was observed for the Vancouver structure. In general, inelastic rotations recorded are small and below 0.02 rad. For one record at one storey, larger value of inelastic rotation was obtained (0.03rad). Braces and columns of MTL14 structure responded elastically. Some limited yielding was observed in bottom storey braces and in one column of VCR14. Results indicate that the duration of inelastic excursion was short with the exception of one record that induced longer inelastic column response (approximately 10s).

As expected, for both structures, the occurrence of the maximum inelastic inter-storey drifts coincided with the maximum inelastic link shear rotations. Both median and 84th percentile values were significantly smaller than those predicted in design, particularly for MTL14 where only about 25 percent of the Code limit was attained.

3.3 Response of the 20-storey frame

The maximum normalized shear forces in links of VCR20 structure exceeded the design limit of 1.3 in six storeys in the upper part of the frame. Maximum median value of 1.47 was observed in the 17th storey. 84th percentile values show the same tendency with slightly increased magnitudes. For Montreal structure, very little yielding was generally observed in links, with the exception of the three upper-storey links where the maximum 84th percentile value reached 1.4 thus slightly surpassing the design limit. The median values of the maximum inelastic shear rotations were below the design limit for both frames. The 84th percentile values exceeded the design limit for Vancouver, reaching 0.103 rad in the 15th storey. For Montreal structures all 84th percentile results were within the anticipated values.

No yielding of outer beam segments was observed for the two frames studied. Some brace yielding was recorded in the bottom and middle-portion braces in Vancouver structure. Columns of all designs responded elastically. The results obtained for the inelastic inter-storey drifts were similar those found for the 14-storey frames. Montreal structure developed less deformations compared to the Vancouver one, but for both design the values remained well within the design requirements.

5. SUMMARY AND CONCLUSIONS

The study addressing seismic design and inelastic response of taller eccentrically braced frames has been presented. 14- and 20-storey frames were designed for Montreal, QC and Vancouver, BC and their inelastic response was studied for different acceleration records. The attempt was made to establish the governing design criteria and propose the most appropriate design sequence. For all structures studied it was found that the selection of link sections was based on the elastic resistance required for the factored load combination with gravity or wind loads and not on the inelastic shear resistance required for seismic load cases. This selection led to the significant overstrength of the links in all frames, particularly in the upper storeys of the frames. Design of other frame members was generally controlled by wind load combinations increasing further the global frame overstrength. Using an increase of the structural mass as indicator it was establish that for Vancouver inelastic inter-storey drift requirements governed frame design while for Montreal ensuring the global frame stability was critical. In spite of the large differences in seismic design base shears, the mass of final designs for the same frame height were almost identical.

Non-linear time history analysis was conducted for selected sets of accelerograms compatible with design spectra at studied locations. For both locations, maximum values of inelastic link shear forces exceeded slightly the values anticipated in design. The magnitude of the strain-hardening obtained in the analysis was consistent with the observations reported in literature. It was found that the ductility demand on links, evaluated through the inelastic shear rotations, was much more important in Vancouver than in Montreal. This was anticipated in view of the fact that the acceleration records in the eastern North-America are dominated by high frequencies. With the exception of the outer beam segments, for which the inelastic behaviour was accepted in design, other members of the frame responded predominantly elastically. However, some yielding of braces and columns was observed in the frames designed for Vancouver. The impact of this behaviour on the frame response will be addressed in the future studies.

6. ACKNOWLEDGMENTS

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REFERENCES

CSA (2005). Limit state design of steel structures. Standard CAN/CSA-S16-01. Canadian Standard Association, Rexdale, Ont.

Koboevic, S.(2000). An approach to seismic design of eccentrically braced frames. *Ph.D. Thesis*. Dept. Of Civil and Mechanical Engineering, McGill University, Montreal.

Mondkar, D.P. and Powell, G.H. (1975). ANSR-1 General Purpose Program for Analysis of Nonlinear Structural response. *Report EERC 75-37*, Earthquake Engineering Research Center, College of Engineering, Univ. of California, Berkeley, Ca.

NBCC (2005) National Building Code of Canada. Associated committee on the National Building Code, National Research Council of Canada, Ottawa, Ont.

Okazaki, T., Arce, G., Ryu, H., and Engelhardt, M. (2005). Experimental study of local buckling, overstrength, and fracture of links in eccentrically braced frames. *ASCE J. of Struct. Eng.* **131:10**, 1526-1535

Ricles, J. M., and Popov, E. P. (1994). Inelastic Link Element for EBF Seismic Analysis. *ASCE J. of Struct. Eng* **120:2**, 441-463.