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STUDY OF GLOBAL BEHAVIOUR OF ECCENTRICALLY BRACED FRAMES IN RESPONSE TO SEISMIC LOADS

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ABSTRACT:

An initial phase of the on-going research project investigating a global seismic response of eccentrically braced frames (EBFs) is presented. The project includes both analytical and large-scale experimental studies. The following objectives are set for the analytical study: (i) determine the most appropriate analytical models to represent seismic behaviour of all frame members; (ii) verify if the current design procedures achieve desired frame response; and (iii) evaluate the impact of the element behaviour non-anticipated in design on global frame performance. This paper describes the work related to objectives (i) and (ii). Two dimensional non-linear time-history analyses were conducted for three-storey chevron-type EBFs designed for typical western and eastern North-American seismic conditions. Special attention was given to the selection of earthquake records and the calibration procedures. Results demonstrate that different analytical models give similar results in terms of maximal element forces but exhibit much higher variability regarding inelastic deformations both at element and global structural level. This may be significant when estimates of inelastic structural deformations are used as the key parameter in design. Preliminary findings also indicate that limited yielding of beams and braces does not seem to have a negative impact on the overall frame performance.

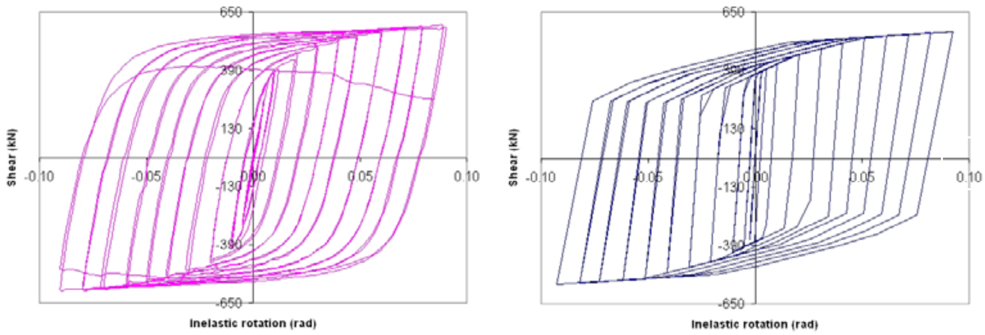
KEYWORDS: steel structures, eccentrically braced frames, global response, non-linear analysis

1. INTRODUCTION

Among traditional steel framing systems used to resist seismic loads, eccentrically braced frames stand out by their effectiveness. They provide an exceptional ductility through inelastic deformations of links, the segments of beams formed between brace connection points, and offer a very high stiffness at working load levels. Design procedures generally assume that the dissipation of energy is exclusive to links and aim to achieve elastic response of all other frame members, while maintaining link deformations bellow the acceptable limits.

Analytical models used to investigate the inelastic frame response under seismic loads are usually constructed to reflect the anticipated behaviour. Links are modelled as inelastic elements with concentrated end flexural and shear hinges and the strain hardening (kinematic or kinematic and isotropic) is included. The multi-linear function that describes the inelastic behaviour of global end hinges is obtained by dividing those into series of sub-hinges (Ricles and Popov, 1994) or by introducing rotational and translational spring elements (Ramadan and Ghodbarah, 1995; Richards and Uang, 2006). As shown in Fig. 1.1, when compared with the latest experimental data on shear links (Okazaki et al., 2005), these analytical models show very good agreement in predicting the maximum shear forces and deformations but the intermediary values are underestimated.

Beams outside of the link, braces and columns, on the other hand, are typically modeled as elastic beam-column elements as no inelastic behaviour is anticipated. Previous studies reported in literature indicate however that elastic response of these frame members is not always achieved. Applying capacity design principles, design forces in these elements are determined on basis of inelastic link resistance, amplified to account for yielding and strain-hardening. Oftentimes, in function of selected geometry of the frame, outer beam segments have to resist high axial forces and bending moments which may require the modification of the beam size. In common case, where the beam section is maintained uniform throughout the span, this is not desirable since the selection of a stronger section entrains even higher forces on all frame members due to increased over-resistance of the



(a) (b)

UTA specimen 4C

Fig. 1.1 Comparison between (a) experimental results Okazai et al. (2005) and (b) analytical results obtained by OpenSees using modeling proposed by Richards and Uang (2006).

link and may lead to less economical designs. This problem can be solved by distributing the link end moment between the beam and the brace, and accepting that beam segment outside the link could respond inelastically. This presumes that the beam-to-breace connection is conceived as moment-resistant. Although previous studies admit that limited yielding of the outside beam segment may be acceptable, little evidence is available regarding the possible extent of that yielding and the impact of this behaviour on overall frame response. The results of non-linear analysis reported also indicated that braces and the columns in the upper tiers of the chevron-type EBFs may develop some yielding, but it is not known how this behaviour influences global frame response and under which circumstances, if any, it may be accepted.

In order to understand better seismic behaviour of EBFs and improve design procedures it is important to advance analytical models so that the possible behaviour of all frame elements under seismic loading can be accounted for. It is also essential to develop further reliable methods to appropriately define earthquake input for linear and non-linear analysis which would combine the latest seismological findings with good understanding of structural response. Such tools would be equally useful in all applications where more realistic estimates of structural deformations are required or where there is a need to quantify anticipated structural damage related to predefined level of seismic loads. This paper presents a study that investigated the modelling of the global seismic response of EBFs. The study was carried out for three- and eight-storey EBFs with following objectives: (i) determine the most appropriate analytical models to represent global seismic behaviour of the frame; (ii) verify if the current design procedures achieve desired frame response; and (iii) evaluate the impact of the element behaviour non-anticipated in design on overall frame performance. This paper describes the work addressing the first two objectives. Procedures and results are shown on the example of the three-storey frame.

2. FRAME DESIGN

Two Canadian locations, Montreal, QC, and Vancouver, BC, were selected for this study, assuming site Class C conditions at the foundation level (360m/s $\leq v_s \leq$ 760m/s). A three-storey EBF structure was designed following the requirements of the National Building Code of Canada (NBCC 2005) and the standard for design of steel structures (CAN/CSA-S16-05). The layout and the framing arrangement are shown in Fig. 2.1.

A parametric study was carried out first to select the optimal link length and the type of connection between the beam and the brace. Designs were evaluated in function of structural weight. Four different lengths were selected for the links (400, 600, 700 and 1000mm) assuming rigid and pinned connection between beam and brace elements. The upper limit on link length was chosen so that the link is shear critical. The structural weight increased in proportion to the link length regardless of the type of connection. Connection type had the largest impact for the frames with the longest link (about 13 percent less weight for the rigid connection compared to the pinned one). Based on this study link length of 600 mm was selected and it was decided to consider brace-to-beam connection as moment-resistant. All other connections were considered as pinned.

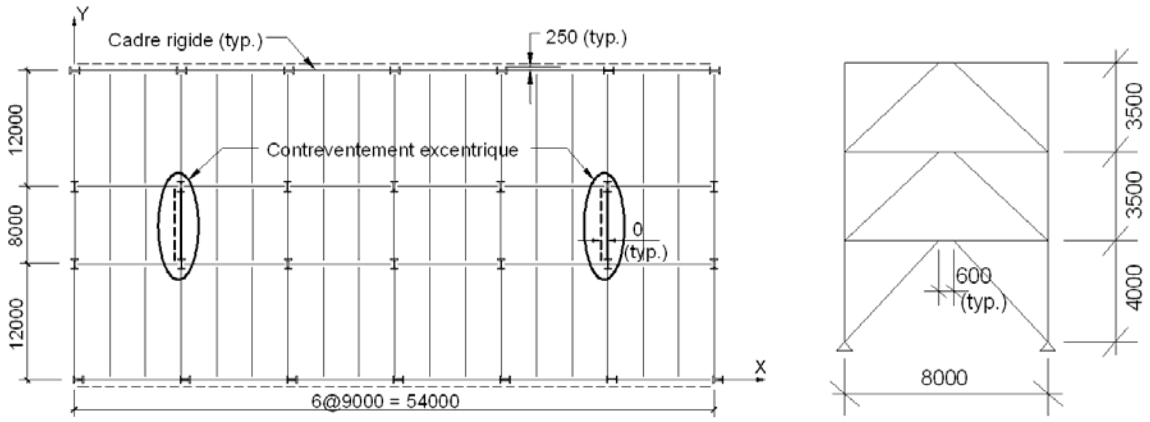


Fig.2.1. Layout and the elevation of the three-storey EBF

The base shear was calculated using the static equivalent force method, employing the Eqn. 2.1:

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

where T_a is the empirical structural period (T_a = 0.025h_n, h_n being 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; I_E = 1.0, and M_v = 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	3 STOREY EBF			
	Montreal (MTL3)		Vancouver (VCR3)	
Seismic weight, *W* (kN)	20363		19991	
Design period, 2T_a (s)	0.55		0.55	
Spectral accelerations at 0.2s and 2s S(*0.2*); S(*2*) (g)	0.69	0.048	0.94	0.17
Spectral acceleration at design period S(2T_a) (g)	0.320		0.609	
Design base shear (% *W*)	5.33		10.15	
Design base shear (kN)	1086		2029	

The design base shear was determined using the augmented empirical period (2T_a) as permitted by NBCC 2005. In subsequent modal analysis it was shown that the fundamental periods of final designs was superior to those assumed during the design process. No change in base shear was made to account for accidental torsion in order to maintain consistency between designed structures and the 2D non-linear analysis performed later. Member sizes were first selected based on ductility design requirements. Links were chosen to have adequate inelastic shear resistance for factored seismic loads including P-Delta effects. As recommended by Popov et al. (1992) the effort was made to maintain the uniform distribution of link resistance-to-link force demand to promote more uniform yielding of links at different storeys. This was not achieved at the top storey were stronger section had to be selected in order to have shear-critical link and Class 1 section.

Following capacity design principles, braces and the outer beam segments were designed for forces introduced by 1.3R_y times the nominal shear resistance of the link, V_p. R_y represents the ratio between the expected and the nominal yield stress and has a value of 1.1. Both elements were treated as beam-columns as the brace-to-beam moment-resisting connection permitted the distribution of link end moments. The floor beam was considered as fully laterally supported, and the combination of bending moment and tensile axial force was critical for these elements. For braces, which were conceived as HSS sections, compressive axial force including gravity loads combined with bending moment governed the design. Distribution of link end moments was initially done in proportion to the relative flexural stiffness of the beam and brace elements, until the portion of the moment assigned to the outer beam segment reached the maximum value which the outer beam segment could carry in combination with tensile axial force. Any remaining moment was than transmitted to the brace as yielding of the outer beam segment was deemed acceptable in design. The assumptions regarding the distribution of link end moment were verified once the frame designs were completed. The columns were assumed continuous over the height and tiered in two-storey segments. Axial forces introduced by the gravity loads were combined with the forces calculated based on the amplified link nominal resistance (1.15R_yV_p for bottom two storeys and 1.3 R_yV_p for the top storey). The allowance was also made for the bending moment resulting from column continuity and the relative storey movements as required by CAN/CSA-S16-05. Inelastic link rotations, γ, were compared to design limit of 0.08rad. In all cases, γ remained bellow the limit reaching the maximum value of 0.07rad.

The stiffness and the strength of the two frames were verified for all relevant load combinations including gravity loads, notional loads, wind and seismic loads. Only top storey columns were slightly increased due to gravity load combinations. The selected shapes for the two frames are presented in Table 2.2.

Table 2.2 Summary of selected shapes (steel CSA-G40.21-350W)

	Structure MTL3 (Mass = 3539 kg, T = 0.80 s)			Structure VCR3 (Mass = 4172 kg, T = 0.62 s)		
Storey	Braces	Columns	Beams	Braces	Columns	Beams
1	HSS203x203x8	W310x79	W250x45	HSS254x254x8	W310x86	W460x60
2	HSS178x178x8	W310x79	W200x31	HSS203x203x9.5	W310x86	W360x39
3	HSS152x152x8	W250x33	W130x28	HSS152x152x9.5	W250x33	W200x27

3. MODELLING FOR DYNAMIC ANALYSIS

3.1 Modelling of frame elements

Analytical models of EBFs studied were constructed using three computer programmes, ANSR-1 (Mondkar and Powell, 1975), DRAIN-2DX (Prakash et al., 1993) and OpeenSees (Mazzoni et al., 2006). The first goal was to represent the inelastic behaviour of shear links that reflected the response observed in the latest experimental studies reported in literature. The second goal was to define more complete models to represent the behaviour of the other frame members so that both elastic and inelastic response could be included. Different analytical models were then compared on basis of elastic and inelastic frame response for selected earthquake records.

In past studies two computer programs were most commonly used to analyse inelastic response of EBFs, namely ANSR-1 and DRAIN-2DX. The capabilities of the two programs for modelling the behaviour of frame members other than links are rather limited. Inelastic beam-column elements are available, which could be used only to represent the cross-section yielding under combined bending moment and axial force. This may be satisfactory for laterally supported outer beam segments for which no lateral-torsional buckling is expected. For braces and columns, lateral-torsional buckling is often a critical design condition.

The main difference between the two programmes lays is the approach to model the inelastic behaviour of the links. ANSR-1 includes a special link element developed by Ricles and Popov (1994). The element contains an elastic beam with plastic hinges concentrated at its ends. 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. Upon yielding, however, interaction between moment and shear is not considered, and the axial deformations are neglected. In DRAIN-2DX, link is modeled by combining an elastic beam with the series of rotational and translational springs having zero length. The inelastic behaviour of each spring, represented by a bilinear force-deformation curve, is combined to represent the yielding response of the whole element.

An analytical model was also developed using the program OpenSees (Fig. 3.1). The links were represented in two different ways. The first model was identical to that used in DRAIN-2DX. To improve the fit between experimental and analytical results over the entire hysteresis, a second model was studied in which the series of springs was replaced with a single spring with inelastic behaviour described using the Giuffré-Menegotto-Pinto (Steel02) hysteretic model. The main improvement however was achieved in the modeling of the behaviour of other frame elements. Outer beam segments and diagonals were modeled using eight nonlinear beam-column elements with fibre discretization of the cross section. In this way, cross-sectional yielding and flexural buckling could be represented. 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 included into the model to account for the end restraint conditions induced by the the gusset plates. Non-linear beam-column elements with Steel02 hysteresis were also used to model column behaviour. Steel02 material was modified as proposed by Lamarche and Tremblay (2008) to account for the residual stresses typical for W-sections. For the same reason, the number of fibres in each element was also increased.

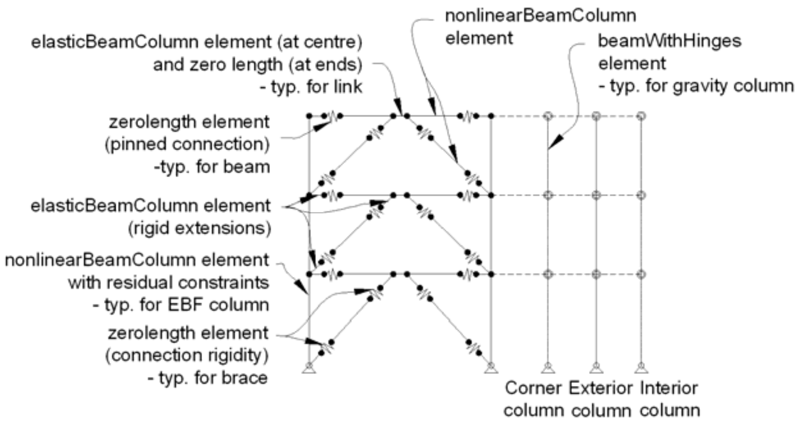
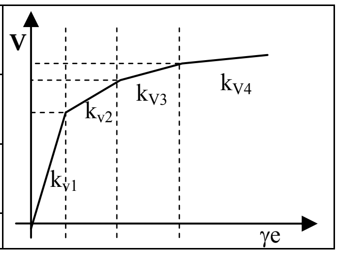


Figure 3.1 OpenSees model of three-storey EBF

The calibration of the link element, applicable to all models, was done using the results of experimental studies conducted by Okazaki et al. (2005) on short shear links, and compared to those used in the previous studies. The OpenSees model of the test setup was built to reproduce the results obtained for 11 specimens. The final values for different yielding points (V_1, V_2, and V_3) represent median values of multi-linear curve derived for each individual specimen. As can be seen from Table 3.1.1, the values obtained are similar to those reported in previous studies. The small differences in V_2 and V_3 values are attributed to the differences in the test steel properties that were used as reference for the calibration in past studies.

Table 3.1.1 Calibration for the link elements

	V_1 [V_p]	k_{v1}	V_2 [V_p]	k_{v2}	V_3 [V_p]	k_{v3}	k_{v4}	
Ramadan et Ghobarah (1995)	1.00*	GA_v/e	1.26*	$0.03k_{v1}$	1.40*	$0.015\ k_{v1}$	$0.002k_{v1}$	
Richards et Uang (2006)	$1.10^{\dagger}$	$2GA_v/e$	$1.30^{\dagger}$	$0.03k_{v1}$	$1.50^{\dagger}$	$0.015\ k_{v1}$	$0.002k_{v1}$	
This study	1.00*	$2GA_v/e$	1.20*	$0.03k_{v1}$	1.35*	$0.015\ k_{v1}$	$0.002k_{v1}$	

* *V based on nominal resistance,* $V_p = \phi\, 0.55 A_v R_y F_y$ *, with* $\phi R_y = 1.0$

† *V based on expected resistance,* $V_p = \phi\, 0.55 A_v R_y F_y$ *, with* $\phi R_y = 1.1$

Preliminary validation for the various EBF frame models was carried out using linear static and modal analysis. Comparison of results obtained for structural periods, element forces and global displacements showed a very good agreement between the different models for the two frames studied.

3.2 Selection and calibrations of earthquake records

Special attention was devoted to the selection and calibration of the earthquake records as exploratory studies had shown great sensitivity of inelastic response parameters, particularly deformations, to different calibration methods. While for the western North-America large database of historical records is available, this is not the case for eastern North-American region. Consequently, historical and artificial earthquake records were chosen to analyse the frames designed in Vancouver and artificial records for those in Montreal.

The initial selection of accelerograms was done based on the combination of magnitude and hypocentral distance scenarios that contribute most significantly to the seismic hazard at the sites. For each location, free-field recordings for firm ground conditions of a moderate event at short-distance and a strong event at long distance were selected. Ten historical records were selected for Vancouver, mostly from the Loma Prieta and Northridge earthquakes. Because these records are typical for California and may differ from those anticipated in Vancouver, 14 artificial records were also selected[1]. For the Montreal site, only 14 artificial records from the same source were used due to the lack of appropriate historical records, typically very rich in high frequencies.

All earthquake records were scaled to represent the level of seismic loads implicit in NBCC 2005. The logical approach is to adjust the accelerograms such that their response spectrum matches the design uniform hazard acceleration spectrum. This was done using different approaches. A subjective approach was used first. The calibration factor was obtained by simple observation of the best fit between the record and design spectra. This was done over a period range determined by the user. That range varied over the record ensemble. This method is simple and effective, but highly dependent on the experience of the analyst. In the equivalent spectral intensity approach, the acceleration record is modified to obtain the same spectral intensity, SI_a, as that of the design spectrum. The integration is done over the same period range for all records. In the Schiff method (Schiff, 1988) the calibration factor is calculated as a product of two values: F_1 which regroups spectra of all records so that the same intensity of the velocity spectra is obtained for intermediate and long periods, and F_2 which anchors the acceleration spectra of the records to the design spectrum so that the same intensity of acceleration spectra is obtained over the range of short periods. Lastly, the authors used a hybrid method that combined the subjective and the equivalent spectral intensity approaches. For a selected record, the range of periods for the calculation of the spectral intensity was defined by the analyst based on the observation of the spectrum and the type of the record. Limits were imposed on the maximum differences in spectral accelerations between the record and design spectra within the chosen period range. For the same range, the record was adjusted such that the sum of the differences in spectral ordinates was approximately equal to zero.

Table 3.2.1 Impact of seismic ground motion calibration methods on inelastic structural response

Record	Calibration method	Calibration factor	V_1 (kN)	V_2 (kN)	V_3 (kN)	γ_1 (rad)	γ_2 (rad)	γ_3 (rad)	Roof displacement (mm)	Base shear (kN)
V15*	Subjective	1.60	1038	674	346	0.029	0.047	0.037	52	2066
	SIa 0.5-4	2.58	1173	780	384	0.116	0.157	0.075	107	2295
	Schiff	3.03	1202	824	399	0.176	0.238	0.102	141	2419
	Hybrid	2.55	1170	779	382	0.114	0.154	0.072	105	2289

** Jan. 17, 1994 Northridge: Station Pacific Palisades-Sunset, M=6.7 R=26 km, PGA=0.197g, PGV=0.149 m/s*

[1] Atkinson, G. 2007. Personal communication.

The impact of the calibration procedure on the inelastic structural response was evaluated using ANSR-1 for the 3-storey frame in Vancouver subjected to two records scaled using each of the four procedures. As can be seen from Table 3.2.1., link deformations and roof displacements were found to be very sensitive to the calibration method employed. The significant differences in the scaling factors obtained for each method did not translate into large variations in the maximum link forces and the base shears. This was anticipated since the forces in this structure are limited by the capacity of the links. The hybrid method was chosen for this study because it combines engineering judgement and a more formal procedure. It also led to results that compare well with the average values obtained using all methods examined.

4. STUDY OF NON-LINEAR RESPONSE

To further compare the performance of different analytical models and validate design procedures, non-linear time history analyses were conducted. Link response was monitored through maximum shear link forces and rotations, while the global structural response was evaluated through inelastic base shear, inter-storey displacements, maximum forces generated in other frame members and the characteristics their inelastic excursions (time and frequency of occurrence and duration). For each accelerogram, the maximum values of the response parameters were found for each storey and the median and 84 percentile values were calculated.

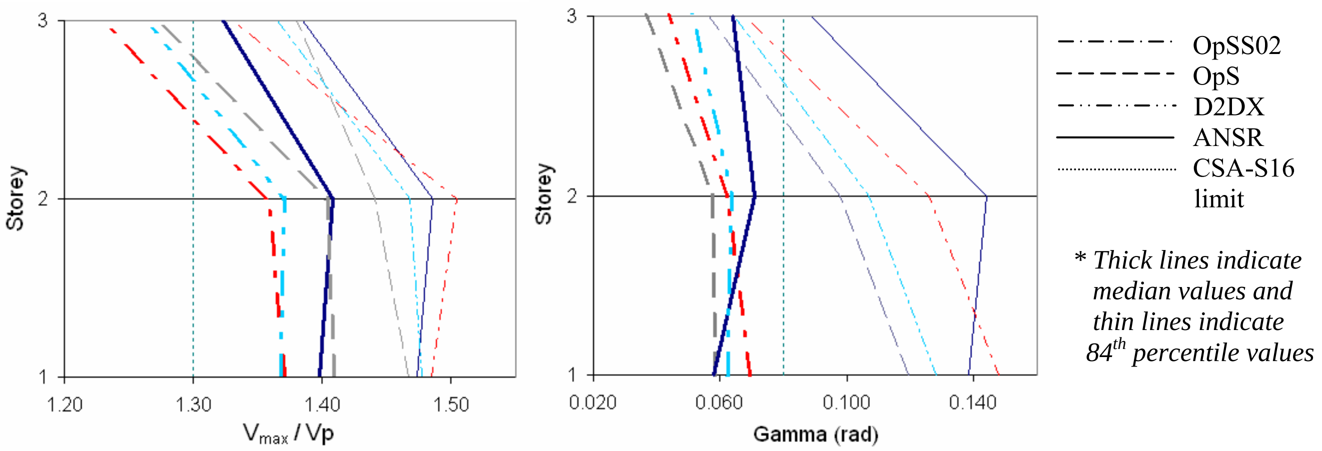


Figure 4.1 Structure VCR3: Median and 84 percentile values of maximum normalized link inelastic shear forces and deformations

All analytical models showed important yielding of two bottom links in the Vancouver structure. As illustrated in Figure 4.1, in both storeys, the median values of maximum link inelastic shear forces exceeded slightly the NBCC limit ($1.3V_p$), ranging between $1.37V_p$ and $1.41V_p$. In general, the values obtained by OpeenSees, with Steel02 link model are the lowest and those obtained by ANSR-1 the highest. Simultaneous yielding of all three links was frequently observed. Links developed important inelastic shear deformations. Maximum median value at all storeys just reached the CSA S16 limit of 0.08rad. The differences are more pronounced when 84 percentile values are compared; ANSR-1 model predicted the maximum value at the second storey, while other models showed the largest rotations at the first storey. Although the location is not consistent, the maximum magnitudes of shear link deformation reached about 0.15rad, which is significantly higher than the code limit.

Median values of base shears in the VCR3 structure varied between 2008kN and 2252kN, which compares well with the design base shear. Slightly larger median values of inter-storey drift index were predicted by ANSR-1 (0.9%), which is still significantly smaller than the code limit of 2.5%. Columns response was essentially elastic. The median values of column bending moments were about 0.2Mp at the bottom two storeys and 0.4Mp at the top storey. This is consistent with the values used in CSA-S16. Some yielding was observed in the outer beam segments and in braces. Analysis run on OpeenSees showed that the yielding was not extensive and did not negatively influence the global frame response. The structures designed for Montreal showed much less yielding of the links and almost no yielding of other frame members. All design limits were respected.

5. SUMMARY AND CONCLUSIONS

The analytical part of a study investigating the global seismic behaviour of eccentrically braced frames (EBFs) with shear links has been presented. The modelling was done using three computer programs: ANSR-1, DRAIN-2DX and OpenSees. The models representing the link inelastic behaviour were calibrated to reflect the results of the latest available test data. For the other frame members, an attempted was made to represent both elastic and inelastic responses. Modelling approaches were compared for three-storey EBFs designed for typical eastern and western North-American locations. Initially, modal and linear static analyses were performed. Comparison of results obtained for structural periods, member forces and global displacements showed a very good agreement between the different models. Further assessment was done using non-linear time-history analysis. Acceleration records were scaled applying a hybrid method proposed by the authors. It was found that, in general, the modeling had less impact on the force response values than on deformation values, both at the global and element levels. This is significant when deformations are used as a key parameter for design. For the Vancouver building, all models yielded link shear forces and deformations higher than those anticipated in design while the inter-storey drift remained well below the design limits. Inelastic response was also observed in beams and braces. Preliminary results obtained from OpenSees suggest that this limited yielding does not have a negative impact on the overall frame behaviour. Detailed evaluation of the global seismic response for EBFs with a more extensive inelastic behaviour of beams and braces is the subject of an on-going study.

6. ACKNOWLEDGMENTS

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