

Seismic Evaluation of Multi-Panel Steel Concentrically Braced Frames

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SUMMARY:

Lateral load resisting systems in tall single-storey steel buildings typically consist of vertical braced frames with two or more bracing panels stacked between the ground and the roof levels. This bracing configuration with bracing members intersecting columns between ground and roof levels raise several concerns regarding their inelastic response under seismic loading. This paper presents a study on the seismic design and nonlinear seismic performance of 3- and 4-panel steel concentrically braced frames designed according to the current Canadian code provisions for steel structures. Dynamic nonlinear time history analysis is conducted on using the Opensees platform to assess the seismic performance of the frames, with particular interest in the seismic force demand imposed on the columns and the buckling behaviour of the columns. The results show that nonlinear response in the frames may not be uniformly distributed over the building height, which leads to high ductility demand in the braces and in-plane bending moments in the columns. No significant out-of-plane bending moments are induced in the columns as a result of brace buckling and yielding.

Keywords: Tall single storey steel building, multi-panel CBFs, nonlinear time history analysis, column stability

1. INTRODUCTION

Concentrically braced frames (CBFs) are among the most popular seismic resistant systems for single-storey steel industrial buildings in North America. For tall buildings, it is common to use multi-panel, or multi-tiered CBFs in which two or more bracing panels, depending on the height of the building, are stacked between the ground and roof levels. Figure 1.1a shows examples of such braced frames. This configuration is preferred to diagonal bracing members continuous from ground to roof level in view of the reduced brace buckling length. For seismic applications, it is also easier with this configuration to meet the brace slenderness limits imposed in the seismic design provisions for minimum energy dissipation in compression.

Based on the capacity design approach for CBFs, beams and columns must be designed for gravity load effects combined with member forces that develop upon brace yielding in tension and buckling in compression. Hence, the selected brace sections directly affect the forces transferred to the beams and columns. Smaller bracing members as typically required in multi-panel CBFs, which may therefore results in smaller beams and columns. Furthermore, since the columns are braced at every panel point in the plane of the frames, the axial compression force capacity of the columns is increased, and smaller sections can be adopted for the columns. Therefore, utilizing a number of X-bracing panels along the height of the building can lead to economical design alternatives for tall single storey buildings such as industrial buildings, airplane hangars, convention centres or warehouse buildings.

As a result of current seismic design philosophy, energy dissipation through inelastic response is constrained to the bracing members, through brace inelastic buckling in compression and brace yielding in tension. All other frame components such as beams, columns, and connections must remain elastic to maintain the integrity of the gravity load-carrying system. In multi-panel CBFs, although it is expected in design that brace buckling and yielding will occur in all panels under the

design base shear, brace tension yielding and significant inelastic buckling will likely take place in only one panel, limiting the storey shear such that yielding will not occur in the other panels, which may lead to concentration of the inelastic demand in that critical panel, as illustrated in Fig. 1.1.

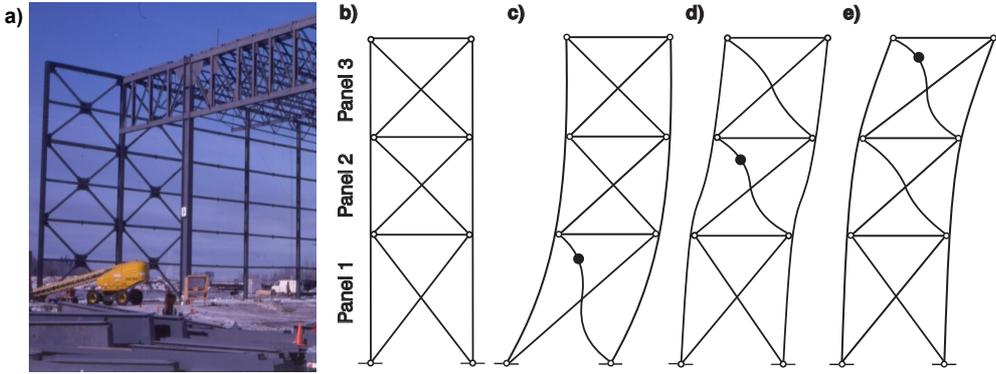


Figure 1.1. a) Tall single-storey steel building with three-panel CBFs along the height; b) Undeformed shape geometry of three-panel CBF; Deformed shape under lateral loading c) Panel 1 is the critical panel; d) Panel 2 is the critical panel; and e) Panel 3 is the critical panel

Non-uniform distribution of inelastic deformations in one of the panels may induce in-plane bending moments in the columns where braces meet the column, impose large inelastic deformation demand on the braces and connections, and cause softening and strength degradation in the critical panel. All these concerns must be considered when completing the design of a multi-panel CBF. The AISC seismic provisions in the U.S. (AISC 2010), Eurocode EC8 (CEN 2004) and the AS4100 Australian standard for steel structures (AS 1998) do not include specific requirements for the seismic design of multi-panel CBFs. Special seismic provisions have been introduced for these multi-tiered braced frames in the 2009 edition of CSA S16, the design standard for steel structures in Canada (CSA 2009). In CSA S16, the system is permitted only for the limited ductility (Type LD) braced frame category for which a ductility-related seismic force modification factor $R_d = 2.0$ is specified. The columns must be designed for the simultaneous action of the gravity loads, and the axial forces and bending moments arising from the braces reaching their probable axial resistances in compression and tension at the design storey drift (δ in Fig. 1.2), assuming brace tension yielding develops in anyone of the bracing panels along the frame height. The columns must also resist out-of-plane bending moments from transverse loads, N , applied at every brace-to-column joints equal to 10% of the compression member forces acting at the column joints. These loading conditions are shown for a three-panel CBF in Fig. 1.2. In the figure, the brace forces C'_u and T_u correspond to the probable post-buckling compressive and yield tensile resistances of the brace. As shown, a horizontal strut must be provided between every panel to transfer the horizontal unbalanced brace loads developing after buckling of the braces. The struts also provide lateral bracing for the columns in the plane of the bracing bent.

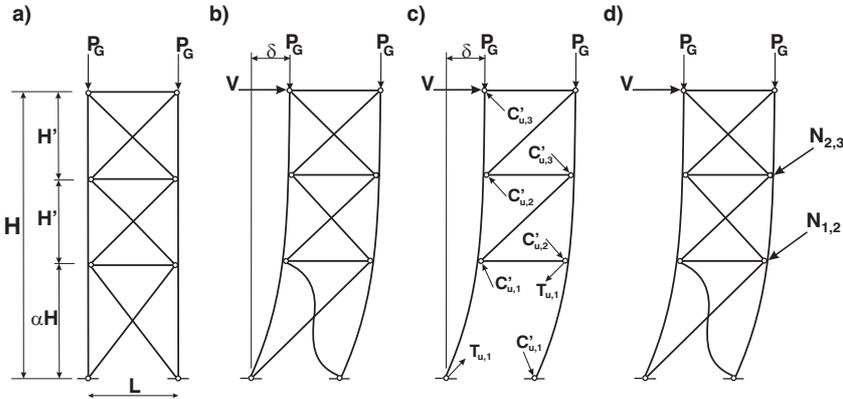


Figure 1.2. Three-panel CBF: a) Frame elevation; b) Lateral deformation; c) Probable brace forces in Panel 1; and d) Column out-of-plane buckling

A preliminary study two-panel CBFs by Imanpour and Tremblay (2012) confirmed the likelihood of inelastic deformation demand along the building height. The study also indicated that two-panel CBFs designed with a force modification factor $R_d = 3.0$ would perform satisfactorily and provided data on in-plane and out-of-plane bending moment demands on the columns. This paper presents a second phase of this study where the seismic design and performance of three- and four-panel CBFs designed with $R_d = 3.0$ are investigated. A value of 3.0 is used for R_d , instead of 2.0 as specified in CSA S16-09, to examine the possibility of using a smaller seismic load for the design of multi-panel CBFs. The ability to correctly predict the location of the critical panel at the design stage is evaluated. Roof drifts, drift demands in the critical panel and in-plane and out-of-plane bending moment demands on the columns are assessed through nonlinear time history dynamic analyses and the results are compared with the design assumptions.

2. DESIGN OF 3-PANEL CBF

2.1. Building studied

A tall single-storey industrial steel building having 128.8 m x 50.4 m plan dimensions was selected for the study. The building height, H , is equal to 18 or 24 m. Four concentrically braced frames with braces intersecting columns along their height are placed in each direction (two CBFs per wall). Each frame is a three- or four-panel X-bracing CBF. The height of the lowest (Panel 1) is equal to αH and the remaining height is equally distributed among the other panels (see Fig. 1.2). A total of sixteen different braced frame configurations were examined to study the effect of the following parameters on the structure seismic response: number of bracing panels, total height of the frame, relative height of the bracing panels, and the span length.

The building is located on a class C site in Vancouver, British Columbia. All frames were designed in accordance with NBCC 2010 (NRCC 2010) and the CSA S16-09 standard. Gravity loads include the design roof dead load (D) and snow load (S) of 1.6 and 1.64 kPa, respectively. The columns of the braced frame studied support 50.4 m roof trusses that span over the full width of the building, resulting in column gravity loads $P_D = 226$ kN and $P_S = 231$ kN. A Ductility-related and overstrength-related seismic force modification factors of 3.0 and 1.3 were used, as specified in NBCC 2010 for moderately ductile (Type MD) CBFs. The equivalent static force procedure was used to calculate the seismic load and accidental torsion was considered to calculate the storey shear resisted by the CBFs.

2.2. Design of braced frame with 3 X-braced panels

The design of a three-panel frame with total height of 18 m is presented herein to illustrate the design procedure and the parameters used to define the forces demand on the columns. The elevation of the frame is shown in Fig. 2.1a. The frame has a width of 5.6 m. The height of Panel 1 is 7.2 m ($\alpha = 0.4$) and the upper two panels (Panels 2 & 3) have identical geometry. The building fundamental period is equal to 0.9 s, resulting in a design spectral acceleration (S) of 0.39 g and a design storey shear per braced frame, V , of 410 kN.

The columns are continuous over the whole storey height and assumed pinned at their both ends. They are oriented such that out-of-plane buckling occurs about their strong axis. Horizontal struts are provided between adjacent panels and at the roof level. They are assumed as pin-connected to the columns. The bracing members are the first CBF components to be designed. They resist the seismic storey shear in tension and compression, so brace axial compression forces are equal to $C_E = 334$ kN in Panel 1 and 285 kN in Panels 2 and 3. Gravity induced compression brace forces of 15, 13, and 13 kN are combined to these seismic effects to obtain the total brace design forces. The braces are designed for compression assuming an effective length of 0.45, taking into account the size and fixity of the brace end connections and the mid-support provided by the intersecting tension braces.

The selected bracing members are shown in Fig. 2.1a. They were selected from square HSS members

conforming to ASTM A500, grade C, ($F_y = 345 \text{ MPa}$). The braces had to meet the lower and upper overall brace slenderness limits of 70 and 200, respectively, to ensure acceptable ductile behaviour, as well as the upper limits on width-to-thickness ratios of the brace cross-section elements, as specified in CSA S16-09. The probable brace resistances in tension (T_u) and compression (C_u) are computed using the equations specified in CSA S16-09 and the probable steel yield strength for HSS members, $R_y F_y = 460 \text{ MPa}$. The probable post-buckling compressive brace resistances, C'_u , are also determined as the brace compression capacity reduces significantly. These values are summarized in Table 2.1. For this frame, three brace force scenarios represent the loading conditions that will exist as the roof displacement is increased: 1) shortly after brace buckling occurs in the panels and brace yielding in the critical panel (Fig. 2.1b), 2) after cyclic inelastic deformations and yielding have taken place in the critical panel, together with brace buckling in the other panels (Fig. 2.1c); and 3) braces reaching their post-buckling resistance in noncritical panels (Fig. 2.1d). The roof displacement corresponding to these three scenarios are, respectively, δ' , δ'' , and δ''' .

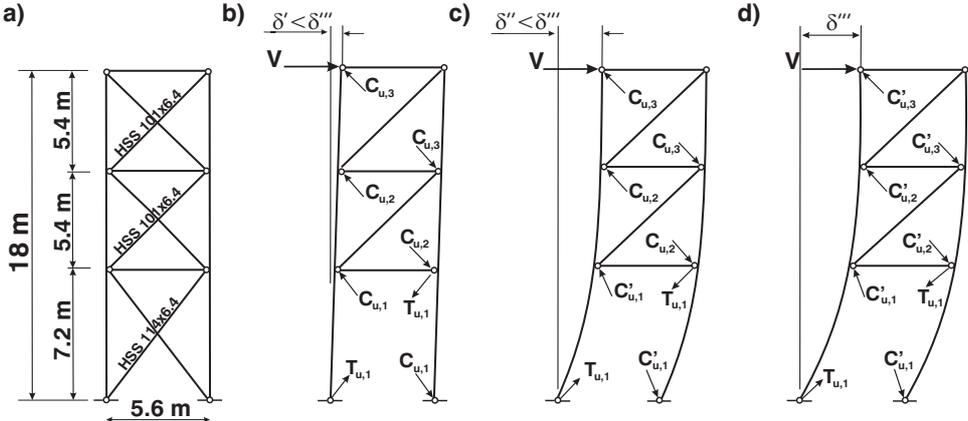


Figure 2.1. a) Selected brace members; Brace force scenarios under seismic loading: b) $C_u + T_u$ in critical panel and $C_u + T$ in noncritical panels; c) $C'_u + T_u$ in critical panel and $C_u + T$ in noncritical panels; and d) $C'_u + T_u$ in critical panel and $C'_u + T$ in noncritical panels

Table 2.1. Properties of the brace members and storey shear resistances per panel

Panel	Shape	A (mm ²)	KL (mm)	C_r kN	C_u kN	C'_u kN	T_u kN
3	HSS 102x102x6.4	2170	3501	328	481	200	998
2	HSS 102x102x6.4	2170	3501	328	481	200	998
1	HSS 114x114x6.4	2480	4105	361	527	228	1141

2.3. Determining the critical panel(s) and design of the column

The determination of the critical, or weakest panel where inelastic deformations are likely to concentrate is required to establish the design brace scenarios of Fig. 2.1 and, thereby, properly assess the demand on the columns. If the critical panel is identified, there is generally no need to consider several loading scenarios associated to other critical panels. As brace tension yielding occurs first in the critical panel, this panel is the one having the lowest shear resistance, V_u . For a given panel, that shear resistance is taken equal to the sum of the horizontal components of the brace forces C_u and T_u , i.e., corresponding to the first brace force scenario in Fig. 2.1b. For the frame example, Panel 1 has lower storey shear resistance ($V_{u,1} = 1024 \text{ kN}$) compared to the other two panels ($V_{u,2} = V_{u,3} = 1065 \text{ kN}$), and is therefore designated as the critical panel. During an earthquake, the maximum anticipated shear force in the frame will be limited to the minimum shear resistance of the critical panel, here Panel 1, and the tension forces in the braces of the other noncritical panels (Panels 2 & 3) will never reach their tensile yield strength.

Although previous study on two-panel CBFs by Imanpour and Tremblay (2012) showed that subsequent inelastic deformations always develop in the same panel where tension yielding initiated, the shear resistance computed based on the two other brace force scenarios should also be examined to verify the

choice of the critical panel. For the second scenario of Fig. 2.1c, the forces in the tension bracing members of Panels 2 and 3 are computed assuming that the braces in Panel 1 reach their yielding and post-buckling probable resistances, while the compression braces in the other panels have buckled and develop their probable compression resistances C_u . For this scenario, the brace tension forces in Panels 2 and 3 are limited to 686 kN, which is smaller than the yield tensile strength of these bracing members ($T_{u,2\&3} = 998$ kN). In the brace force scenario of Fig. 2.1d, the brace post-buckling condition is attained in all panels so that the probable post-buckling compressive resistances, C'_u , is assigned to all compression braces. In that case, the resulting force in the tension braces of Panels 2 and 3 is equal to 968 kN, which is less than the brace yield load $T_{u,2\&3} = 998$ kN, indicating that brace tension yielding will very likely only occur in Panel 1.

When performing capacity design in accordance with CSA S16-09, storey shears in a braced frame need not exceed the storey shear determined with seismic loads computed with $R_d R_o = 1.3$. For the frame studied, this storey shear is equal to 1197 kN, which is higher than the shear resistance of the critical panel: $V_{u,1} = 1024$ kN. In this case, this upper bound does not apply. If it governs, brace forces will be limited and brace tension yielding and, perhaps, brace buckling will not occur in the bracing members of the critical panel. This situation must be considered in design.

Following capacity design principles, the columns and struts must be designed using the brace axial forces that will developed upon brace yielding and buckling, and all three brace force scenarios described in Fig.2.1 must be considered. The third scenario shown in Fig. 2.1d governs the design of the horizontal struts. This scenario corresponds to the brace post-buckling resistance (C'_u) in the compression members, which leads to larger axial compression strut loads when compared the other scenarios. For this example, the maximum axial force occurs in the strut above Panel 1 (557 kN), and a HSS 152x152x7.9 was selected to resist this force. The columns are designed to resist the axial force induced by gravity loads together with the forces being induced by the bracing members. Moreover, special design requirements including in-plane bending moments determined from the expected inelastic panel deformation pattern and transverse notional loads at joints must be considered. The maximum axial compression force of 2875 kN due to seismic effects occurs in the column of Panel 1 in scenario 1. The gravity load induced compression is added to these seismic effects for the load combination $E + D + 0.25S$, which leads to a total designed factored axial compression force of 3159 kN.

The column design is therefore an iterative process as the moment of inertia of the columns must be known to determine the design in-plane bending moments being induced by a given frame lateral deformation pattern. A column section is first selected based on the computed axial compression only and this trial column section is then used to estimate the frame lateral deformations that are subsequently considered to determine the column in-plane bending moments. This in-plane bending moment together with the out-of-plane bending moment induced by the transverse notional load based on the compression members (column, strut, and compression brace) acting at the joints are used to design the columns. As indicated, Panel 1 is the critical panel for the frame example; the lateral deformation of the frame is calculated considering the large lateral displacement induced in this panel, and elastic lateral displacement of the other panels. The total inelastic displacement at the roof level could take place when the bracing members in the panels reach their buckling or post-buckling resistances based on the braced frame configurations and member selection. For the frame example, the total inelastic displacement at roof is determined from the elastic roof displacement, δ_e , as prescribed in NBCC 2010, and equal to $R_d R_o \delta_e = 3.0 \times 1.3 \times 31 = 120$ mm ($= 0.67\%$ H). This value exceeds δ''' , which means that all three scenarios of Fig. 2.1b to d must be considered. It is noted that the elastic frame lateral displacement, δ_e , includes the contribution of the axial deformations of all columns and all bracing members.

Once the total inelastic displacement at the roof level is known, the corresponding lateral deflection at the top of each panel is determined under the storey shear of the critical panel in the post-buckling range, $V'_{u,1}$, as the storey shear in noncritical panels (Panels 2 and 3) are limited to the shear resistance of Panel 1 when the roof deflection δ''' is reached. To calculate the lateral deflection of each panel, it is assumed that the total plastic deformations take place in the critical panel. If the critical panel is located in identical panels, or all of the panels are identical, the plastic deformations are equally shared between the

identical panels. For the frame example, the value of the deformation at top of Panel 1 to 3 is equal to 72, 95, and 120 mm respectively. In-plane bending moments in the column at the points where the braces meet the column can be computed using the absolute lateral deformation of each panel. The classical three-moment equation developed for assessing bending moments on continuous beams on multiple supports subjected to relative settlements is used for the calculations. To assemble the three-moment equations for the frame example, the relative lateral displacement between two adjacent panels is treated as a support settlement, as shown in Fig. 2.2, and two sets of equations are assembled, one for spans AB & BC, and one for spans BC & CD:

$$M_A H_1 + 2M_B(H_1 + H_2) + M_C H_2 = \frac{6EI_c(\delta_1)}{H_1} - \frac{6EI_c(\delta_2 - \delta_1)}{H_2} \quad (2.1)$$

$$M_B H_2 + 2M_C(H_2 + H_3) + M_D H_3 = \frac{6EI_c(\delta_2 - \delta_1)}{H_2} - \frac{6EI_c(\delta_3 - \delta_2)}{H_3} \quad (2.2)$$

In these equations, EI_c is the flexural stiffness of the column, and H_1 , H_2 , and H_3 are the height of the panels, as shown in Fig. 2.2. By solving these two equations, the column weak axis bending moments at the brace intersecting points can be obtained, and the maximum column weak axis bending, $M_{cy,max}$, is then determined (Fig. 2.2). As prescribed in CSA S16-09, the columns must be checked for simultaneous axial compression, in-plane bending moment (M_{cy}), and out-of-plane bending moment (M_{cx}). Out-of-plane, strong axis bending moment is generated by the transverse loads (N) applied at each brace-to-column intersecting point, and the maximum value is obtained at the point of application of the largest load (Fig. 1.2). These transverse loads are equal to 10% of the forces in the compression members meeting at the intersecting points. For the frame studied, the compression force in the column, braces and struts at the two intersecting points along the column height are used to compute the two transverse point loads. The calculated in-plane and out-of-plane bending moments for the final section selected for the column, W610x307, are tabulated in Table 2.2. It is noted that the in-plane bending moment, $M_{cy,max}$, does not co-exist with the maximum axial force, C_f , and the maximum out-of-plane bending moment, $M_{cx,max}$, as the moment M_{cy} develops in the post-buckling range whereas the maximum C_f and M_{cx} forces occur at buckling of the compression braces.

Table 2.2. Calculation of the column in-plane and out-of-plane bending demand

Panel (i)	Column Section	H_i mm	δ_i mm	M_{cy} kN-m	M_{cy}/M_{pcy}	N_{brace} kN	N_{column} kN	N_{strut} kN	N_i kN	M_{cx} kN-m	M_{cx}/M_{pcx}
3	W610x37	5400	120			481					
				13.0	0.03				274		
2	W610x37	5400	95			481	1924	331			
				67.0	0.14				399	1998	0.58
1	W610x37	7200	72				3159	354			

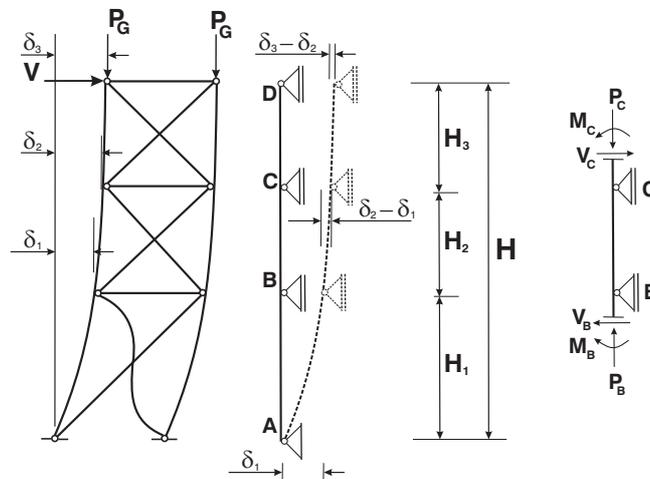


Figure 2.2. Calculation of the in-plane flexural demand on the column with the three-moment equation for the case where Panel 1 is the critical panel

In Table 2.2, the column factored in-plane and out-of-plane bending moments are $M_{cy,max} = 67$ kN and $M_{cx,max} = 1998$ kN. The column is then verified using the CSA S16-09 interaction equations for members subjected to axial compression and biaxial bending:

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0 \quad (2.3)$$

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0 \quad (2.4)$$

where $U_1 = \omega_1 / (1 - C_f / C_e)$, $\beta = 0.6 + 0.4 \lambda_y \leq 0.85$, $M_{fx} = M_{cx,max}$, $M_{fy} = M_{cy,max}$, and C_f is the column factored axial compression. As prescribed in CSA S16, Equation 2.3 is used to verify the capacity of the column for cross-sectional strength, overall member strength in the plane of the frame and lateral-torsional buckling strength out of the plane of the frame. Table 2.3 presents key results from Equation 2.3 and 2.4. The column section also satisfies the requirements of Class 1 as specified in CSA S16 to prevent section local buckling.

Table 2.3. Calculation of the column (W610x307) in-plane and out-of-plane bending demand

Limit State	C_r kN	ω_{1x}	C_{ex} kN	U_{1x}	ω_{1y}	C_{ey} kN	U_{1y}	β	M_{rx} kN-m	M_{ry} kN-m	ratio
cross-sectional strength	12172	0.85	17302	1.04	0.60	9177	1.00	0.85	3083	699	0.89
overall member strength	8608	0.85	17302	1.04	0.60	9177	0.91	0.85	3083	699	1.00
lateral-torsional buckling strength Equation (2.4)	9422	0.85	17302	1.04	0.60	9177	0.91	0.85	2945	699	0.99
									2945	699	0.77

The loads C_{ex} and C_{ey} are determined with buckling lengths equal to H and αH , respectively. The value of C_r is computed using $\lambda = 0$ for cross-sectional strength, is equal to the lesser of C_{rx} and C_{ry} for the overall member strength, and is equal to C_{rx} for lateral-torsional buckling strength. C_{rx} and C_{ry} are determined based on the buckling lengths of H and αH , respectively, but effective slenderness factors K_x and K_y smaller than 1.0 are used to account for the fact that the axial load varies along the column height. For the frame example, $K_x = 0.81$ and $K_y = 0.68$ were used. A similar column design procedure is used if the critical panel is located in a panel other than Panel 1.

The column deformation profile, and thereby, the seismic demand induced in the columns and struts of the braced frame is affected by the location of the critical panel over the height of the frame. It is therefore crucial to properly identify the critical panel, and possible uncertainties should be taken into account in that process. In particular, when the frame includes two or more identical bracing panels, one should verify if differences in member end conditions may affect the selection of the critical panels. If this is not possible, all potential critical panel scenarios should be examined in the design. The variability in the brace probable resistance used to define the storey shear resistance, including the variability in steel yield strength or the effects of the high strain rates on the buckling resistance of bracing members, may change the location of the critical panel and, thereby, the column demand. Particular attention must be paid to these uncertainties when the demand of the column is computed.

3. NONLINEAR TIME HISTORY ANALYSIS

3.1. Braced frame model

Nonlinear time history analysis was performed to assess the seismic performance of multi-panel CBFs. A total of sixteen multi-panel CBFs were designed using the described design procedure. Numerical

models of these frames were created using the Opensees program (McKenna and Fenves, 2004). The bracing members and the columns were modeled using force-based beam-column elements with fiber discretization of the cross-section to distribute inelasticity. A corotational formulation was considered to account for geometric nonlinearities (Crisfield, 1991). The uniaxial Giuffr -Menegotto-Pinto (Steel02) steel material with kinematic and isotropic hardening was used to simulate Bauehinger effect under cyclic loading (Aguero et al., 2006; Uriz et al., 2008). The nominal yield strength $F_y = 345$ MPa was assigned to the steel material and residual stresses were specified for the columns. Additional detail is given in Imanpour et al. (2012).

The frames studied include 8 three-panel and 8 four-panel CBFs, as both configurations represent the possible design alternatives for tall single storey steel buildings. Two different frame widths of 5.6 and 8.0 m, two total heights equal to 18 and 24 m, and two height ratios $\alpha = 0.33$ and 0.40 for three-panel CBFs, and $\alpha = 0.25$ and 0.30 for four-panel CBFs, were examined. In the analyses, the gravity loads were applied first on the top of the frame columns, and nonlinear time history analysis was subsequently performed using a series of 10 ground motion records scaled to match the design spectra for Vancouver.

3.2. Prediction of the critical panel(s)

Figure 3.1a and b show the mean plus one standard deviation of the panel drifts and of the drift ratios computed for every panel of the studied frames. The panel drifts are obtained by dividing the relative panel lateral displacements by the respective panel height and the drift ratios are computed by dividing the drift of the critical panel by the drift of the noncritical panels. As shown in Fig. 3.1, there is a more pronounced drift in the critical panel than in other panels, which verifies the concentration of the nonlinear deformation under seismic loading for the multi-panel CBFs. This behaviour results in high ductility demand in the braces of the critical panel. The study on 3- and 4-panel CBFs resulted in a different lateral behaviour for multi-panel CBFs compared to the observations previously made for 2-panel CBFs (Imanpour et al., 2012; Imanpour and Tremblay, 2012): in the 3- and 4-panels CBFs with a critical panel being located in one of the identical panels, there are at least 2 identical critical panels and inelastic deformations do not concentrate in a single panel as the other identical panels may also contribute to nonlinear response by yielding and buckling of their braces. This sharing of inelastic demand is caused by the shear forces developing in the columns due to in-plane bending, these shear forces being sufficient to trigger brace yielding and buckling in the adjacent identical panels, the columns should also have sufficient stiffness to develop the shear forces. For a 3- and 4- panel CBF with a critical panel being located in one of the identical panels, the column shear force required to initiate inelastic response in the bracing members of an adjacent identical panel is obtained by static analysis of the frame:

$$V_{c-3P} = \frac{(C_{ui} - C'_{ui}) \cdot (H_i/L)}{4 + (1 - \alpha)/(1 + \alpha)} \quad (3.1)$$

$$V_{c-4P} = \frac{(C_{ui} - C'_{ui}) \cdot (H_i/L)}{4 + 4(1 + 2\alpha)/(7 + 17\alpha)} \quad (3.2)$$

where H_i is the height of an identical panel (i), and L is the frame width. Columns must be designed to resist the shear force from Equation (3.1) for 3-panel CBFs, and from Equation (3.2) for 4-panel CBFs so that the yielding and buckling will take place in the adjacent identical panel during the inelastic seismic response.

It should be noted that the concentration of the nonlinear response may occur in either one of the identical panels if a frame has two or more identical panels, or in the panel which was defined as a noncritical panel at the design stage, because, the cyclic loading could affect on the brace resistance in compression, particularly, when the storey shear resistance of the panels are close. Further study is required to demonstrate the effective factors on the brace resistance, and develop a more accurate design method to predict the critical panel and thereby the bending demand on the column.

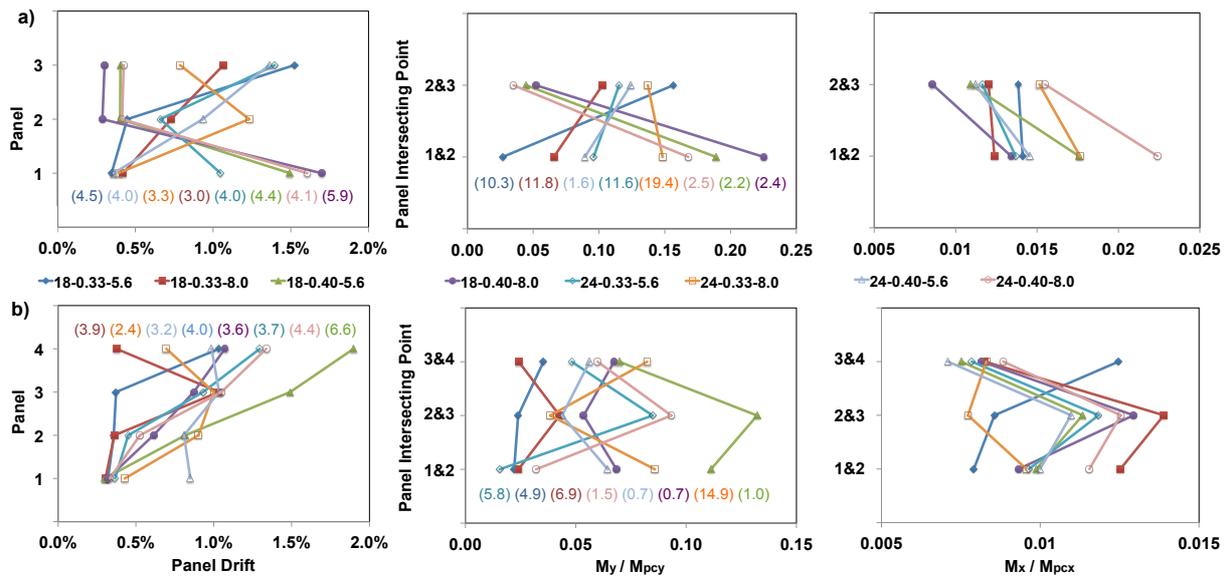


Figure 3.1. Mean plus one standard deviation of panel drifts, in-plane bending moments in the columns, and out-of-plane bending moment in the columns for a) 3-panel CBFs, and b) 4-panel CBFs

3.3. Bending moment demand on the columns

For the ensemble of records, the mean plus one standard deviation of the in-plane and out-of-plane bending moments induced at the brace intersecting points in the columns are presented in Fig. 3.1a and b for 3- and 4-panel CBFs respectively, the ratio of the computed (maximum value along the height) to design bending moments are also presented for each frame in these figures. Concentration of the inelastic response in the braces of the critical panel resulted in non-uniform distribution of the lateral deformation, which, in turn, induced in-plane bending of the columns. This bending demand varies for the studied frames. It is maximum for the point below or above the critical panel; however, these values are noticeably different from the bending moments used in design. In-plane bending moments are generally higher for 3-panel CBFs, due to the more non-uniform lateral deformation of the 3-panel CBFs compared to the 4-panel CBFs. In Fig. 3.1, very limited out-of-plane column bending moments are computed for 3- and 4-panel CBFs, varying between 0.01 and 0.025 M_{pcx} , much less than the large out-of-plane bending moments considered in design. The out-of-plane bending demand is maximum at the same point where the largest in-plane bending moments are observed, suggesting that interaction exists between in-plane bending demand and out-of-plane response (Imanpour et al., 2012). As shown, in-plane and out-of-plane bending moments are more pronounced for the frames with unequal panel heights compared to the frames with identical panel dimensions over the building height, thus confirming the effect of different panel height ratios on the column demand.

3.4. Assessing the special seismic provisions in CSA S16-09

The nonlinear time history analysis showed that the location of the critical panel where the inelastic deformation is concentrated is mostly the weakest panel computed according to the storey shear resistance, neglecting the variability in brace probable resistances. As shown in Fig. 3.1, on average, the in-plane bending demand of the column is 3 to 5 times higher than the value considered in the design, and conversely, the out-of-plane bending demand is much lesser (35 to 60 times) than the value applied to design of the column by the notional transverse load specified in CSA S16-09. The findings of this study are in agreement with the results of a previous study performed on two-panel CBFs (Imanpour & Tremblay, 2012), which suggests that the provisions of CSA S16-09 regarding the seismic design of columns with braces intersecting between floors in multi-panel CBFs do not reflect the nonlinear response of the multi-panel CBFs and should be revised and modified to include a more realistic and accurate design procedure.

4. CONCLUSIONS

The seismic response of 16 three- and four-panel CBFs was studied using nonlinear time history analyses to evaluate the influence of the frame geometry on the seismic behaviour. The frames were designed based on the specification of CSA S16-09 with a ductility-related force modification factor, $R_d = 3.0$. One design example was presented. The conclusions can be summarized as follows:

- Nonlinear deformations of the frame as well as inelastic response are not uniformly distributed over the frame height and tend to concentrate in the critical panel(s).
- Non-uniform distribution of the lateral displacements over the frame height induces in-plane bending moments in the columns; this demand was found to be higher than the bending moments calculated according to the seismic design procedure of CSA S16-09.
- The concentration of lateral displacements along the frame height and the resulting in-plane bending demand on the columns do not appear to induce significant out-of-plane bending moments in the columns; however, in all frames studied, maximum in-plane and out-of-plane bending moments occur at the same time and at the same location along the column height.
- The seismic provisions of CSA S16-09 regarding the design of columns with braces intersecting between floors in multi-panel CBFs should be reassessed and modified to include a more realistic and accurate design procedure.
- The determination of the critical panel(s) is critical step in the determination of brace axial force scenarios and the demand on the columns demand. The influence of the variability in brace tensile and compressive resistances, the end restraint conditions, and the dynamic loading effect on material strength should be taken into consideration in the development of an accurate method to predict the location of the critical panel(s) and, thereby, column seismic demand.

Detailed finite element analyses are needed to assess the stability and strength of the columns in multi-panel CBFs under lateral loading based on the column seismic demand obtained in this study.

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REFERENCES

- Aguero, A., Izvernari, C., and Tremblay, R. (2006). Modelling of the Seismic Response of Concentrically Braced Steel Frames using the OpenSees Analysis Environment. *Int. J. of Advanced Steel Construction*, **2:3**, 242-274.
- AISC. (2010). *ANSI/AISC 341-10, Seismic Provisions for Structural Steel Buildings*. AISC, Chicago, IL.
- AS. (1998). *AS4100, Steel structures*, Standards Australia, Sydney, Australia.
- CEN. (2004). *Eurocode 8, EN 1998-1:2004: E, Design of structures for earthquake resistance*. Comité Européen de Normalisation (CEN), Brussels, Belgium.
- CSA. (2009). *CSA-S16-09, Design of Steel Structures*. Canadian Standards Association, Mississauga, ON.
- Crisfield, M. A. (1991). *Non-linear finite element analysis of solids and structures*, John Wiley & Sons, West Sussex.
- Imanpour, A., Tremblay, R., and Davaran, A. (2012). Seismic performance of steel concentrically braced frames with bracing members intersecting columns between floors. *7th STESSA 2012*, Santiago, Chile, 447-453.
- Imanpour, A., and Tremblay, R. (2012). Analytical Assessment of Stability of Unbraced Column in Two Panel Concentrically Braced Frames. *3rd International Structural Specialty Conference*, Edmonton, Alberta, Canada, Paper No. 1218.
- McKenna, F. and Fenves, G.L. (2004). Open System for Earthquake Engineering Simulation (OpenSees). Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, CA. (<http://opensees.berkeley.edu/>)
- NRCC. (2010). *National Building Code of Canada 2010, 13th ed.* National Research Council of Canada, Ottawa, ON.
- Uriz, P., Filippou, F.C., and Mahin, S.A. (2008). Model for Cyclic Inelastic Buckling of Steel Braces. *J. Struct. Eng.*, ASCE, **134:4**: 619-628.