Improving the Seismic Stability of Concentrically Braced Steel Frames

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This paper was presented at the 2004 Annual Stability Conference of the Structural Stability Research Council.

Past analytical studies have shown that multi-story concentrically braced steel frames (CBFs) are prone to concentration of inelastic demand and soft-story response under strong seismic events (Redwood, Lu, Bouchard, and Paultre, 1991; Martinelli, Perotti, and Bozzi, 2000; Sabelli, 2001; Tremblay, 2000, 2003; Tremblay and Poncet, 2005). A similar behavior was also observed in past test programs on multi-story braced steel frame specimens (Ghanaat, 1980; Foutch, Goel, and Roeder, 1987; Uang and Bertero, 1986). This response can be attributed to the poor hysteretic behavior of steel bracing members, typically characterized by a severe degradation of the compressive resistance and an accumulation of permanent elongation, and the limited capacity of braced steel frames to redistribute the inelastic demand over the height of the structure. Furthermore, the high inelastic demand resulting from large story drifts can lead to brace fracture and dramatic reduction in story shear resistance (Foutch et al., 1987; Uriz, 2005).

Story mechanism is more likely to develop in tall braced frames due to more pronounced higher mode responses and larger P-delta effects, and this behavior can eventually lead to dynamic instability. Building height limitations have been introduced in the Canadian standard for the design of steel structures, *Limit States Design of Steel Structures*, hereafter referred to as CSA-S16-01, to mitigate this phenomenon (CSA, 2001). Structures exceeding the prescribed height limits can still be used, provided that the design engineer can demonstrate that the structures will exhibit a stable inelastic response when subjected to design level earthquakes.

In this paper, different solution schemes are examined to guard against soft-story response and instability in multi-story braced steel frames: (1) reducing the inelastic demand level by increasing the design seismic loads, in other words, lowering the force modification factor used in design; (2) using a response spectrum dynamic analysis method in design to achieve a more uniform demand/supply ratio over the building height; (3) employing bracing members with stable inelastic cyclic response; and (4) using a special dual bracing configuration that encourages the vertical distribution of the inelastic demand over the building height. The benefits of these methods are evaluated and compared by means of incremental nonlinear dynamic analyses performed on sample building structures varying from 4 to 16 stories in height.

STRUCTURAL SYSTEMS

Three different braced steel frame systems were considered in this study. The first system is a conventional concentrically braced frame (CBF) designed to dissipate energy through tension yielding and inelastic buckling in the bracing members (Figure 1a). The other two systems consist of chevron braced frames built with buckling-restrained braces (BRBs). The regular BRB frame system (Figure 1b) includes only buckling-restrained braces. In the dual buckling-

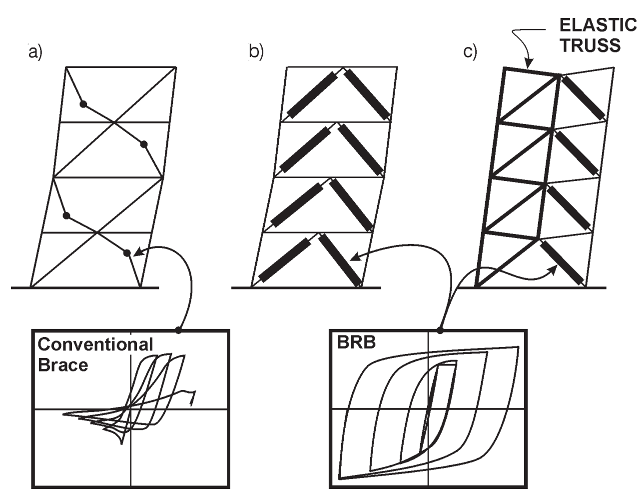

Fig. 1. Structural systems and brace hysteretic responses: (a) conventional CBF; (b) buckling-restrained braced (BRB) frame; (c) dual buckling-restrained braced (dual BRB) frame.

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restrained braced frame concept shown in Figure 1c, buckling-restrained braces are provided only on one half of the bracing bent while the other half consists in a vertical truss designed to remain essentially elastic under strong ground motions. In all three cases, non-moment-resisting beam-to-column joints are used in the braced frames and the building gravity load carrying system, and the lateral loads are assumed to be entirely resisted by the bracing bents.

Conventional CBFs

A split-X (or two-story X) configuration was chosen for the conventional concentrically braced frame structures. This bracing arrangement has been shown to exhibit similar inelastic seismic behavior compared to chevron-bracing with strong beam design while requiring less steel tonnage because the unbalanced post-buckling brace force demand on beams where two braces at one level meet at mid-span of the beams is mostly resisted by the tension-acting brace in the adjacent level. In addition, story shears in split-X bracing do not induce axial deformations in beams, which results in bracing bents that are stiffer than comparable chevron bracings.

The two concentrically braced frame categories defined in the Canadian seismic provisions were examined in this study: Type MD (Moderately Ductile) CBFs and Type LD (Limited Ductility) CBFs. The main difference between the two categories is the value of the ductility-related force modification factor, R_d, that is used as a divider in the calculation of the seismic loads: R_d is equal to 3.0 and 2.0, respectively, for Type MD and LD braced frames. Because Type MD CBFs are designed for lower earthquake loads, they are expected to undergo larger inelastic deformations under the design earthquake. Despite this difference, similar detailing rules are prescribed in CSA-S16-01 for the two systems. For instance, maximum brace slenderness or maximum width-to-thickness ratio for brace cross-sections apply to both braced frame categories when tension-compression braces are used, as is the case for the split-X configuration adopted in this study. For both categories, brace connections as well as beam and column members in the bracing bents must be sized according to capacity design principles; in other words, they must be designed for the member forces that will develop under strong earthquakes when the braces reach their expected compressive or tensile strength. In CSA-S16-01, the use of Type MD CBFs is limited to buildings up to eight stories in height, unless stable inelastic response can be demonstrated. For Type LD braced frames, the height limit is extended to 12 stories in view of the lower inelastic demand expected in the braces and, thereby, the reduced likelihood of soft-story response.

The R_d factor in the 2005 *National Building Code of Canada* (NRCC, 2005), hereafter referred to as 2005 NBCC, ranges from 1.0 for brittle systems such as plain, unreinforced masonry, to 5.0 for the most ductile systems such as ductile moment-resisting steel frames. The corresponding values of the response modification coefficient, R, in ASCE/SEI 7-05 (ASCE, 2005) are 1.5 and 8.0, respectively. In ASCE 7-05, the special steel concentrically braced frame category qualifies for an R factor of 6.0 and is allowed in buildings up to 49 m in height (= 12 stories assuming 4 m story height) for Seismic Design Categories D and E.

Buckling-Restrained Braced Frames

Buckling-restrained (or unbonded) braces are typically made of a steel plate with a dog-bone shape that is inserted in a steel tube filled with concrete (Watanabe, Hitome, Saeki, Wada, and Fujimoto, 1988; Iwata, Kato, and Wada, 2000; Clark, Kasai, Aiken, and Kimura, 2000; Ko, Mole, Aiken, Tajirian, Rubel, and Kimura, 2002). An unbonding material is used between the steel core and the concrete fill. The steel plate is then prevented from buckling and can yield both in compression and tension, which leads to a much more stable hysteretic response with superior energy dissipation capacity compared to ordinary bracing members that buckle in compression (Figure 1b). A chevron bracing configuration was adopted for the two BRB frames. This arrangement has been used in practice (for example, Ko et al., 2002). It has also been shown to perform well in full-scale tests (Tremblay, Degrange, and Blouin, 1999; Lin, Weng, Tsai, Hsiao, Chen, and Lai, 2004; Mahin, Uriz, Aiken, Field, and Ko, 2004; Fahnestock, 2006) or analytical studies (Sabelli, Mahin, and Chang, 2003; Uang and Kiggins, 2003; Kim and Choi, 2006).

For the regular BRB system (Figure 1b), two buckling-restrained braces are used at each level, and yielding is expected to develop nearly simultaneously in both braces. Hence, although individual bracing members can exhibit a stable cyclic inelastic behavior, soft-story response can easily develop as the all-pinned structure possesses limited capacity to trigger simultaneous brace yielding in adjacent floors after initiation of inelastic brace response at a given level. Only the continuity of the columns over two or three stories, as typically encountered in multi-story structures, can provide some restraint against variations in the story drift demand from one story to the next (Tremblay and Stiemer, 1994).

The dual buckling-restrained braced frame in Figure 1c was proposed by Tremblay (2003) as a means for mobilizing the entire energy dissipation potential of the structure. The bracing bent includes two vertical trusses: one in which energy is dissipated through inelastic response of buckling-restrained braces, and one in which the members are designed to behave elastically. The role of the elastic truss is to maintain similar story drift angles in adjacent levels during inelastic response and, thereby, prevent soft-story response. A simple geometry was adopted with only one buckling-restrained brace per floor, and the elastic truss is built from single diagonal members connected to the beams and a vertical column member located at the center of the bracing bent.

BUILDINGS STUDIED

Geometry and Loads

The seismic performance of all three braced frames was examined for building structures having the plan framing layout shown in Figure 2a. Four different building heights were considered: 4, 8, 12, and 16 stories. The structures were assumed regular in plan and in elevation. Their response was investigated in the direction parallel to the short walls and, as shown, resistance to lateral loads in that direction is provided by two identical braced frames located along the perimeter (dotted lines in the figure). This configuration was adopted as it results in a higher design lateral to gravity load ratio compared to the case of bracing bents located along interior column lines and, hence, represents a more critical case in terms of seismic demand.

In-plane accidental torsion was neglected in the study. The seismic loads were then equally distributed between the two braced frames and a 2D analytical model could be used for the analysis, as illustrated in Figure 2b for the 4-story split-X CBF system. The model included one braced frame together with its tributary leaning gravity columns and rigid floor diaphragm response was assumed such that all columns experienced the same lateral displacements. The story height at every level, h_s, was set equal to 3.8 m (12.5 ft) and two-story column segments were employed.

The buildings were assumed to be located in Vancouver, B.C., Canada, and the design was performed according to the 2005 NBCC and the CSA-S16-01 Standard. The gravity dead loads, D, floor live load, L, and roof snow load, S, are given in Figure 2a. Earthquake loads are discussed in the next section. In the calculation of the column axial loads, the floor live load was multiplied by the reduction factor $F_r = 0.3 + (9.8/B)^{0.5}$, where B is the cumulated tributary floor area. When considering only gravity loading, the factored loads were obtained from the two combinations: $1.25D + 1.50L + 0.5S$, and $1.25D + 0.50L + 1.50S$. Load combinations involving earthquake loads, E, were: $1.0D + 0.5L + 0.25S + 1.0E$ or $1.0D - 1.0E$, the latter being used when earthquake and gravity load effects were opposed. Wind effects were assumed not to govern the design and were neglected.

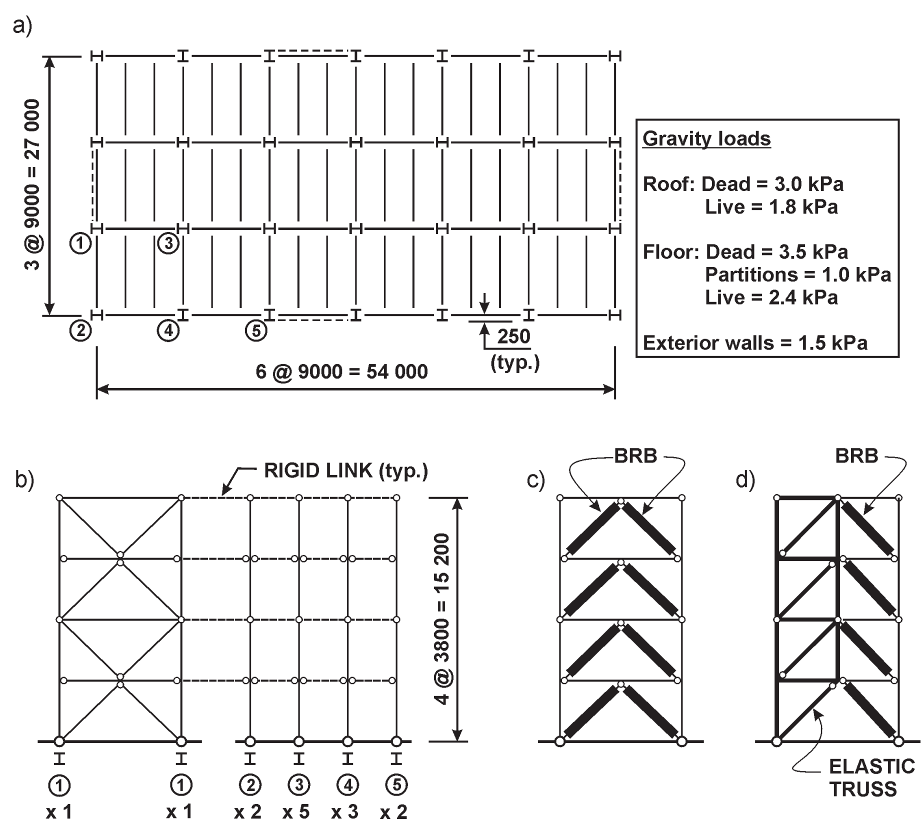

Fig. 2. Buildings studied: (a) floor plan view; (b) analytical model of the MD and LD braced frames; (c) elevation of the BRB frame; (d) elevation of the dual BRB frame.

Seismic Design Base Shear Force

In NBCC 2005, the lateral seismic design force, V, is given by

$$V = \frac{S(T_a)\,M_v\,I_E\,W}{R_o\,R_d} \leq \frac{2}{3}\frac{S(0.2)\,I_E\,W}{R_o\,R_d} \tag{1}$$

where $S(T_a)$ is the design spectral response acceleration at the design fundamental period, T_a, M_v is a factor that accounts for higher mode effects on base shear, I_E is the importance factor, W is the seismic weight, and R_d and R_o are, respectively, the force modification factors related to the ductility and the overstrength of the structural system.

The design spectrum S is determined from 2% in 50 years uniform hazard spectral acceleration ordinates specified for the site, $S_a(T_a)$, and the soil class. It is equal to $F_aS_a(0.2)$ for $T_a < 0.2$ s, the smaller of $F_vS_a(0.5)$ and $F_aS_a(0.2)$ at $T_a = 0.5$ s, $F_vS_a(1.0)$ at $T_a = 1.0$ s, and $F_vS_a(2.0)$ at $T_a = 2.0$ s. For periods between 0.2 and 2.0 s, S is obtained by linear interpolation. For $T_a > 2.0$ s, a minimum seismic load determined with S at $T_a = 2.0$ s must be considered. In these expressions, F_a and F_v are, respectively, the acceleration-based and velocity-based site coefficients. For Vancouver, S_a values of 0.94, 0.66, 0.33, and 0.17g were used at periods equal to 0.2, 0.5, 1.0, and 2.0 s, respectively. Firm ground condition (Site Class C) was assumed in the study, with F_a and $F_v = 1.0$.

For design, the fundamental period is equal to $T_a = 0.025h_n$, where h_n is the total building height. Alternatively, T_a can be taken as the period obtained from methods of mechanics but the period used in design cannot exceed two times the value given by the empirical expression ($T_a < 0.05h_n$). For braced frames built in the western part of Canada, the parameter M_v is 1.0 for $T_a \leq 1.0$ s and 1.5 for $T_a \geq 2.0$ s. For intermediate period values, the product $S(M_v)$ is obtained by linear interpolation. All buildings were considered to be of the normal importance category with $I_E = 1.0$. For the entire building plan area, the seismic weight of the roof level and the floor levels were, respectively, 5640 kN (1270 kips) and 6930 kN (1560 kips).

Dynamic versus Static Analysis Procedures

In 2005 NBCC, a dynamic analysis procedure must be used to determine member forces and deflections due to earthquakes unless certain criteria are met in terms of site spectral accelerations at the site, structural irregularity, building height or periods, etc., in which case an equivalent static force procedure is permitted. When dynamic analysis is used, the modal response spectrum method is generally adopted by practicing engineers as the method is implemented in most computer-aided programs. The seismic actions are calculated using the design spectrum S and the results are multiplied by the ratio V_d/V_e, where V_d is the design base shear and V_e is the base shear obtained from the modal analysis. For regular structures such as those studied herein, the force V_d is equal to the lesser of $V_e\,I_E/(R_oR_d)$ and 80% V. For irregular structures, the lower bound on V_d is increased to 100% V in view of the more complex response anticipated for these structures.

For regular structures less than 60 m in height and having $T_a < 2.0$ s, the equivalent static force procedure can be used in lieu of the dynamic analysis. In the static approach, the lateral loads, F_x, applied at each level of the structure are given by

$$F_x = \frac{(V - F_t)\,W_x\,h_x}{\sum_{i=1}^{n} W_i\,h_i} \tag{2}$$

In this expression, F_t is a concentrated lateral load at the building top, and W and h are, respectively, the seismic weight and the height from the base of the level under consideration. The force F_t is equal to $0.07T_aV$ but need not exceed $0.25V$.

The lateral deformations obtained from the dynamic or static elastic analysis methods must be multiplied by R_oR_d/I_E to give realistic estimates of the anticipated deflections. For buildings of the normal importance category ($I_E = 1.0$), the so-computed story drifts are limited to 0.025 h_s.

Cases Studied

Table 1 summarizes the main design parameters for the 15 cases considered in the study: the structural system, the R_d factor used, the type of analysis [S = static analysis, D = dynamic (modal response spectrum) analysis], whether or not P-delta effects were considered in design (Y = yes, N = no), the design period, T_a, the design base shear ratio, V/W, and the computed periods in the first two modes of vibration, T_1 and T_2.

The Type MD conventional CBF system is used for all building heights. The design was first performed using the equivalent static force procedure and without consideration of P-delta effects. This design is then used as the reference case for comparison purposes. The potential for enhanced seismic performance resulting from increased lateral building capacity was then examined for the 12- and 16-story buildings by considering P-delta effects in the design of the Type MD frames and, subsequently, by using the Type LD CBF system. For these four structures, the design was also performed using the equivalent static force procedure.

For the 12- and 16-story buildings, the computed fundamental period was found to be longer than 2.0 s and the Type MD CBFs were therefore redesigned using the modal response spectrum analysis method, as required by the 2005 NBCC. For the 12-story structure, P-delta effects were also considered in the dynamic analysis procedure. Finally, the two buckling-restrained braced frame systems were consid-

Table 1. Design Parameters, Properties and Confidence Level Against Global Collapse for the Frames Studied										
n	Type[a]	R_d	Analysis Procedure	*P*-Δ Effects	T_a[b]	*V/W*[c]	T_1	T_2	Steel[d]	L_{CC}[e]
levels					s	%	s	s	tons	%
4	MD	3.0	Static	N	0.74	12.6	0.75	0.32	6.8	99.9
8	MD	3.0	Static	N	1.52	6.33	1.58	0.56	18	98.4
12	MD	3.0	Static	N	2.28	4.36	2.55	0.84	35	78.8
	MD	3.0	Static	Y	2.28	4.98	2.39	0.79	39	89.9
	LD	2.0	Static	N	2.24	6.54	2.24	0.74	44	99.0
	MD	3.0	Dynamic	N	2.28	3.49	2.89	0.93	29	Failed
	MD	3.0	Dynamic	Y	2.28	4.07	2.62	0.86	33	84.3
	BRB	4.0	Static	N	2.28	3.27	3.06	1.07	36	50.7
	Dual BRB	4.0	Static	N	2.28	3.27	2.83	0.95	37	97.7
16	MD	3.0	Static	N	3.04	4.36	3.08	1.03	72	83.6
	MD	3.0	Static	Y	3.04	5.07	3.03	0.97	78	94.0
	LD	2.0	Static	N	2.88	6.54	2.85	0.93	89	95.9
	MD	3.0	Dynamic	N	3.04	3.49	3.77	1.18	52	71.9
	BRB	4.0	Static	N	3.04	3.27	3.37	1.19	106	54.7
	Dual BRB	4.0	Static	N	3.04	3.27	3.21	1.04	105	97.5

[a]MD = Moderately ductile; LD = Limited ductility.
[b]T_a = period used in design.
[c]V_d/W for frames designed using the dynamic analysis procedure.
[d]Steel tonnage per braced bay.
[e]Level of confidence against global collapse.

ered for the building with 12 and 16 stories. For these four BRB frames, static analysis was used and P-delta effects were neglected.

As indicated, the R_d factors for Types MD and LD braced steel frames are, respectively, 3.0 and 2.0 in the 2005 NBCC. For both CBF categories, the overstrength-related factor, R_o, is equal to 1.3. Conversely, no R_o and R_d values are specified in the 2005 NBCC for BRB frames. For these frames, $R_o = 1.3$ was deemed representative of the dependable overstrength resulting from the use of the material resistance factor $\phi = 0.90$ in design, the difference between nominal and expected steel yield strengths, and the strain hardening response exhibited by typical buckling-restrained braces. An R_d factor of 4.0 was adopted for the two BRB systems in view of the superior inelastic response of the braces compared to traditional bracing members (Tremblay, Poncet, Bolduc, Neville, and DeVall, 2004; Tremblay, Bolduc, Neville, and DeVall, 2006).

Design Strategies

An iterative design procedure was used for all frames, the period T_a at each step being set equal to the computed period of the structure designed in the previous step. The process was halted when convergence was achieved on the period or when T_a reached the limit of 2.0 times the empirical value. As shown in Table 1, the second condition governed in all cases except for the two Type LD CBFs. However, limiting the design period had negligible impact on the lateral resistance of the structures: for the 4- and 8-story frames, T_a and T_1 are nearly equal, and T_a is longer than 2.0 s for the 12- and 16-story structures, a region of the spectrum where *S* remains constant.

When P-delta effects were included in design, the first order story shear forces, V_x, were multiplied by an amplification factor $(1 + \theta_x)$, where $\theta_x = (R_o R_d\,\Delta)/(V_x\, h_s)$ and Δ is the story drift from first order elastic analysis. Brace sizes were then increased accordingly, as well as beams and columns due to capacity design rules. The same approach was used for both the dynamic and the static analysis procedures. Figures 3a through 3d show the story shears used in each frame design. Figure 3e presents the variation of θ_x over the building height for the 4-, 8-, 12-, and 16-story Type MD CBFs designed with the static analysis method as well as for the 12-story Type MD CBF designed according to the dynamic analysis procedure.

For a given system, the base shear ratios for the 12- and 16-story frames are the same because the earthquake design force *V* must remain constant for periods longer than 2.0 s. Because ground motion spectral accelerations typically reduce with the building period, the 16-story structures possessed higher lateral resistance relative to the expected earthquake demand when compared to the 12-story frames. The 16-story structures designed with the static equivalent force procedure were also designed for relatively higher story shears along their height as the force, F_t, for these structures was larger than that used for 12-story buildings (F_t increases with T_a). As shown in Figure 3, P-delta effects are small (θ_x less than 0.1) for the 4- and 8-story Type MD frames and, hence, were neglected in design. For the 12- and 16-story structures, accounting for P-delta effects increased the design story shears and, thereby, the lateral frame resistance, by up to approximately 20% and 30%, respectively. As shown, using a Type LD CBF with $R_d = 2.0$ instead of 3.0 resulted in a further increase in lateral forces.

For the Type MD frames designed with the response spectrum analysis method, the design base shear, V_d, was limited to $0.80V$, which resulted in reduced design seismic loads and, thereby, a lower lateral resistance for these structures compared to the same frames designed with the equivalent static force procedure. Moreover, story shears at intermediate levels from dynamic analysis were also relatively lower than those obtained from the static method. Due to the higher R_d value used in their design, the two BRB frames have smaller lateral strength compared to the Type MD frames.

Minimum weight design was performed for all members. Although this approach may not lead to the most economical solution for actual structures, it gives the minimum steel tonnage required when applying a given design method, which allows direct comparison of the impact of the various methods on steel requirements. In addition, minimum weight designed structures are more likely to represent the most critical case when evaluating the appropriateness of design assumptions through structural performance assessment. For the conventional CBFs, the steel braces were made of ASTM A500 Gr. C square tubing [F_y = 345 MPa (50 ksi)]. Brace design was governed by compression under combined gravity and earthquake loads, as well as by the *KL*/*r* and *b*/*t* limits imposed in CSA-S16-01 for ductile behavior. An effective length factor, $K = 0.9$, was adopted in the calculations. For the BRB systems, the core of the braces was made of ASTM A36 steel [F_y = 248 MPa (36 ksi)] and an effective brace area equal to 1.46 times the core cross-section area was used to account for transition and connection material at the brace ends. W-shapes made of ASTM A992 steel [F_y = 345 MPa (50 ksi)] were used for the beams and columns of all frames.

Deflections were verified after completion of the strength design. The anticipated design story drifts were found to slightly exceed the $0.025h_s$ limit for the 16-story Type MD structure designed with the static method without consideration of P-delta effects and for the 16-story regular BRB frame. In both cases, the structures were stiffened by increasing the size of the columns in the lower levels of the bracing bent, thus without introducing additional story shear resistance in the frames such that the relationship between the design lateral loads and the seismic performance was

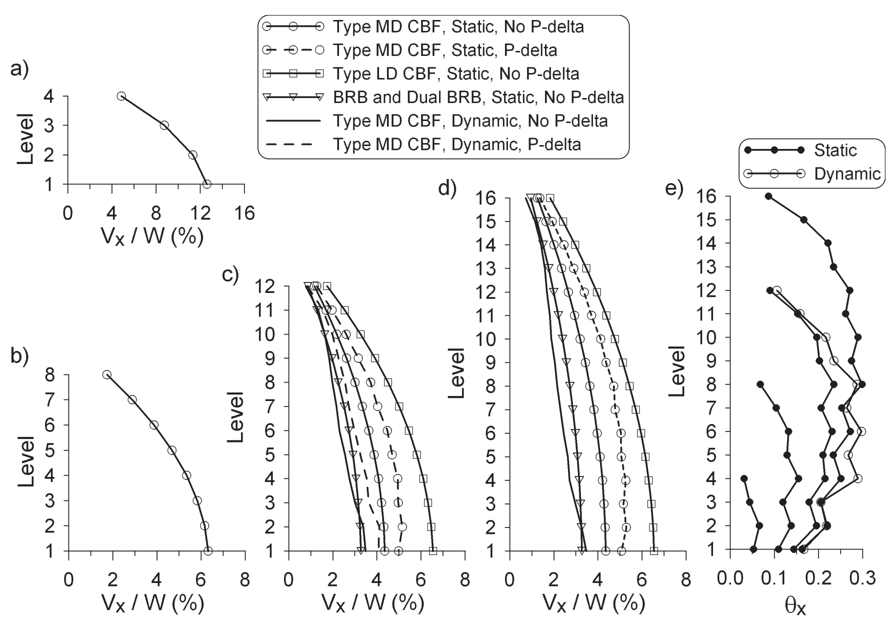

Fig. 3. Design story shear forces: (a) 4-story frame; (b) 8-story frame; (c) 12-story frames; (d) 16-story frames; (e) stability coefficients.

not influenced by drift limit requirements. For the BRB frames, axial deformations in the beams of the chevron configuration that was adopted contributed to the larger story drifts experienced by the BRB structures compared to the split-X bracing. A more effective design probably could have been achieved for the 16-story BRB frame if a two-story X-bracing geometry had also been used with the buckling-restrained braces.

Design Approach for the Dual BRB Frames

The same seismic loads were used for the two BRB systems and, hence, the buckling-restrained braces selected for the regular BRB frames could also be used in the corresponding dual BRB structures. The same exterior columns could also be used as the overturning moment in the dual system is limited by the capacity of the buckling-restrained braces. Axial forces in the diagonal members and the center column of the elastic truss largely depend on the tendency of the structure to form a story mechanism under inelastic response. Early work on the system (Tremblay, 2003) showed that these forces typically increase with the level of inelastic demand. Therefore, they cannot be determined directly from elastic analysis, either static or dynamic, and an iterative design process involving nonlinear dynamic analysis is needed to properly size these members for a given ground motion hazard level.

In this study, a simplified method was adopted to size the elastic truss members so that the potential of the structural system for improving the seismic stability of tall buildings could be examined. One member size was used for all braces of the elastic truss. The selected shape was designed to develop twice the yield capacity of the BRB members located at the building first level. Similarly, the center column was set with a uniform cross-section over the entire building height. As shown in Figure 4, that column was selected to sustain at the second topmost floor the compression force anticipated when buckling-restrained braces yield in compression and the diagonal members of the elastic truss reach their tensile yield strength. It must be realized that this approach is somewhat conservative and arbitrary and does not necessarily lead to the optimum solution for a given building. This had some impact on the structure response, as discussed later when analyzing the results.

Steel Tonnage Requirements

Table 1 gives the total steel tonnage required for the columns, beams, and braces of one bracing bent for each structural system. Centerline dimensions were used to evaluate the weight of the W-shapes and the tubular braces. For the BRB members, the steel material required for both the core plates and the exterior restraining tubes was included. The core plates were assumed to be cut from a plate 2.5 times wider than the width of the brace core segment and the length of the exterior tubes was set 1 m shorter than the center line brace dimensions.

Comparison between the different design strategies can be made for the 12- and 16-story buildings using the Type MD CBF designed with the static analysis procedure and without P-delta effects as the reference case. The amount of steel material increases by approximately 10% when including P-delta effects and by 25% when using R_d = 2.0 instead of 3.0 (Type LD versus Type MD). Conversely, adopting the spectral analysis procedure reduced the steel tonnage by 18% and 28% for the 12- and 16-story buildings, respectively. This drop can be mainly attributed to the 20% reduction in earthquake design forces and the relatively lower story shear force demand along the building height associated with the dynamic analysis procedure. The weight of the 12-story regular BRB frame compares well with that of the Type MD reference frame. A relatively much larger amount of steel was needed, however, for the 16-story BRB frame to meet the code drift limits. The 12- and 16-story dual BRB frames required marginally more steel compared to the regular BRB frames.

ANALYSIS

Modeling Assumptions

Nonlinear time step dynamic analysis of the structures was performed using the Drain-2D computer program by Kanaan and Powell (1973). The numerical models included one of the two bracing bents acting in the direction parallel to the

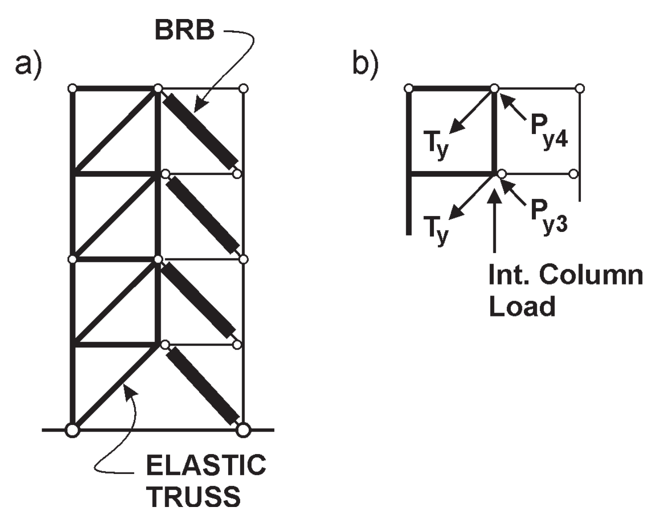

Fig. 4. Design of the interior column in the dual BRB frames: (a) dual BRB system; (b) column design load.

short walls and the tributary leaning gravity columns. The three different models are shown in Figures 2b to 2d for a 4-story building. The leaning gravity columns are only illustrated in Figure 2b but they were also included in the BRB and dual BRB frame models.

All conventional bracing members, including the braces of the elastic truss of the dual BRB systems, were modeled using the inelastic brace buckling element with pinned ends developed by Jain and Goel (1978) (element no. 9 of this reference). The initial and post-buckling compression resistances of these elements were set equal to $C_u = AF_y (1 + \lambda^{2n})^{-1/n}$ and $C'_u = AF_y (a + b\lambda^{-c}) \leq C_u$, respectively. In these expressions, A is the brace cross-section area, F_y is the yield strength, λ is the brace slenderness parameter $(= (KL/r)(F_y/\pi^2 E)^{0.5})$, with KL/r being the brace effective slenderness with $K = 0.9$, $E = 200\,000$ MPa, $n = 1.34$, $a = 0.084$, $b = 0.12$, and $c = 1.61$ (Tremblay, 2002). Buckling-restrained braces were modeled using truss bar elements exhibiting bilinear hysteretic axial response with 2% strain hardening. The columns were made of frame elements with bilinear plastic hinges concentrated at their ends. They were assumed continuous over two stories with pinned connection splice connections. Beam members were also modeled using frame elements with bi-linear plastic hinges at their ends. As indicated, beam-to-column joints in split-X and regular BRB frames were assumed to be pin-connected.

A Newmark constant acceleration integration scheme with a maximum time step of 0.0005 s was used. P-delta effects were considered in the calculations with 100% of the dead load, 50% of the floor live load, and 25% of the roof snow load applied to the structure. Rayleigh damping was used with 5% of critical damping in the first two vibration modes, as commonly used for this type of analysis (Uang and Kiggins, 2003; Sabelli et al., 2003).

Incremental Dynamic Analysis Procedure

An incremental nonlinear dynamic analysis procedure was adopted to assess and compare the seismic performance of the various structural models. The earthquake record ensemble adopted in the analysis included four simulated and six historical ground motion time histories produced by intra-plate seismic events matching the two dominant magnitude-hypocentral distance scenarios for the Vancouver region: **M**6.5 at 30 km and **M**7.2 at 70 km (Tremblay and Atkinson, 2001). The amplitude of the ground motions was first adjusted to match, on average, the design spectra over the applicable period range. The 5% damped response spectra of the scaled artificial and historical records are compared to the design spectrum in Figure 5. In the incremental dynamic analysis, this ground motion amplitude corresponds to a scaling factor of 1.0.

Incremental dynamic analysis (IDA) was performed by applying a scaling factor to the ground motions that varied from 0.2 to 3.0 in successive increments of 0.2. For each ground motion and scaling factor, the maximum value of the peak story drift ratios ($\%h_s$) over the building height was retained and plotted against the scaling factor. Figure 6 shows an example of one set of 10 individual IDA curves com-

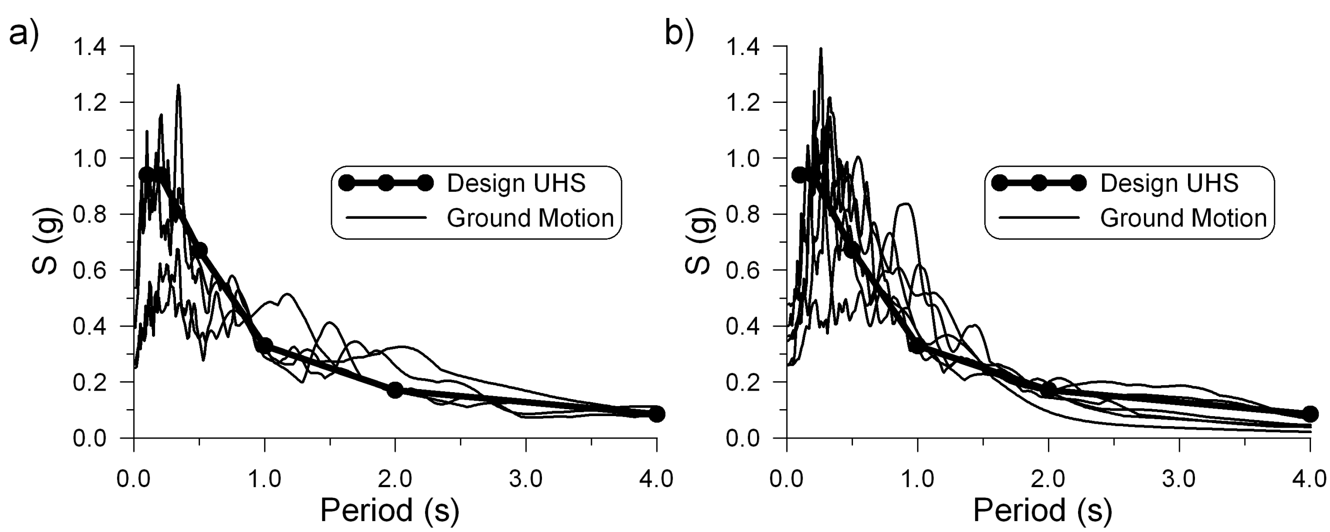

Fig. 5. Design UHS and scaled (SF=1.0) ground motions 5% damped acceleration spectra: (a) simulated time histories; (b) historical records.

puted for a particular structure, as well as the corresponding 16%, 50%, and 84% percentile IDA curves that were subsequently obtained by joining the percentile quantities computed at each scaling factor ordinate (Vamvatsikos and Cornell, 2002a, 2002b). In the next section, the 50% percentile IDA curve is used to assess and compare the performance of the structures.

The results of the incremental analyses were also used to determine the level of confidence against structural collapse, L_{CC}, for each of the structural systems. The calculations were performed following the FEMA 350 procedure developed for steel moment-resisting frames (FEMA, 2000; Cornell, Jalayer, Hamburger, and Foutch, 2002; Yun, Hamburger, Cornell, and Foutch, 2002). This confidence level is based on the factored lateral demand to capacity ratio of the structure and the change in ground motion level with annual probability of exceedance at the site. The method also accounts for the variability in structural demand and uncertainties inherent in the evaluation of the demand and the capacity. A similar approach was used by Uriz (2005) and a detailed description of the method that was followed herein can be found in Tremblay and Poncet (2005).

Analysis Results

Figure 7 shows the variation of the 50% percentile IDA curve with the building height for Type MD conventional CBFs. The 4- and 8-story frames developed a similar robust performance under ground motions with incrementally increasing amplitudes well above the design level, whereas much lower lateral capacity is exhibited by the CBFs that have 12 and 16 stories. It is noted that the two taller structures were designed for the same *V/W* ratio (Table 1), as the floor base shear for periods longer than 2.0 s applied to both structures, but the demand from the ground motion time histories was relatively lower for the 16-story building due to its longer period. The difference in performance between the lower 4- and 8-story frames when compared to the taller structures is confirmed by the L_{CC} values given in Table 1. Confidence levels against collapse of 78.8% and 83.6% were obtained for the 12- and 16-story buildings, respectively, which is lower than the 90% acceptance criteria recommended in FEMA 350. Excellent performance with L_{CC} close to 100% was achieved for the 4- and 8-story structures.

Figure 8 shows the influence of increasing the lateral resistance of the structures by amplifying the story shears for P-delta effects in design or by using a Type LD system. In both cases, the higher lateral capacity resulted in improved seismic response and delayed dynamic instability. The L_{CC} values in Table 1 indicate that either method is fine to reach a satisfactory performance against collapse prevention. In Figure 8, the computed median IDA curves for the Type MD frames designed with P-delta effects and the Type LD CBFs are nearly the same, in spite of the fact that the design story shears were quite different for these two structure types (Figure 3d). For the particular buildings studied here, it happened that the same brace cross-sections were used at several floors of the two frames because the members were selected on a minimum weight basis and the choice of available steel shapes that met the code *KL/r* and *b/t* limits was limited. It is expected that, under other circumstances, poorer response

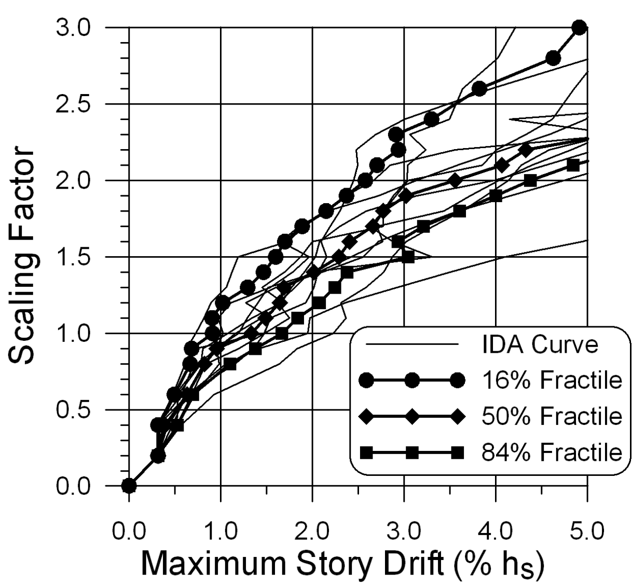

Fig. 6. Individual IDA curves and corresponding percentile curves for the Type MD 4-story braced frame.

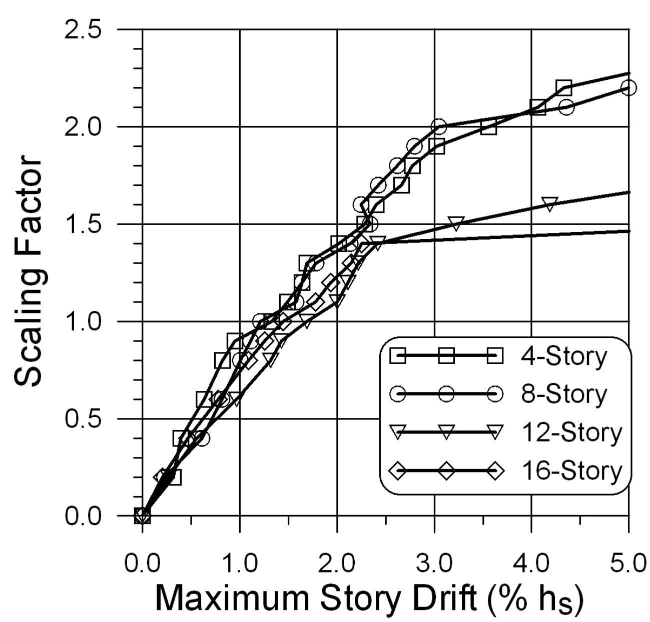

Fig. 7. Influence of building height on the 50% percentile IDA curve for Type MD braced frames (static analysis, no P-delta effects in design).

could be obtained for the Type MD CBFs designed with P-delta effects compared to the Type LD frames.

As discussed earlier, building height limits of 8 and 12 stories were introduced in CSA-S16-01 for Type MD and Type LD CBFs. In addition, the 2005 NBCC requires that P-delta effects be included in design and one possible approach to meet this requirement consists of applying a θ_x story shear amplification factor as was done in this study. The IDA results in Figures 7 and 8 seem to indicate that CBFs designed in accordance with these provisions would perform adequately. In the supplement to CSA-S16-01 published in 2005 (CSA, 2005), the height limits for Type MD and LD frames were changed to 40 and 60 m, respectively, which approximately correspond to 10 and 16 stories assuming a story height of 3.8 m. Beyond 32 m (8 stories), however, the design lateral strength of Type MD frames must be linearly increased up to a value corresponding to R_d = 2.4 for a building height of 40 m. A similar rule also applies for Type LD frames taller than 48 m (≈ 12 stories). Based on the analytical results obtained herein, these newly adopted requirements should also result in sufficient protection against structural collapse.

Figure 9 compares the response of Type MD CBFs designed with the static and dynamic methods of analysis. The results clearly show that the modal response spectrum method does not lead to a more stable behavior, even if this analysis procedure is mandatory for such tall buildings. This is likely caused by the 20% seismic load reduction that is permitted when the response spectrum analysis method is employed. For the 12-story CBF designed without P-delta effects, dynamic instability occurred under two ground motions scaled at the design level (scaling factor = 1.0). No L_{CC} value was computed for this building. In Table 1, it is shown that amplifying the story shear forces from dynamic analysis to account for P-delta effects in design was not sufficient to bring the L_{CC} parameter above the 90% minimum level for that building. An unacceptable confidence level (71.9%) was also obtained for the 16-story Type MD CBF designed with the dynamic analysis procedure. Further studies are needed to examine if better performance could be achieved if the 20% reduction was omitted in design when applying the dynamic analysis procedure.

The performance of the two BRB systems is compared to that of the Type MD CBFs in Figure 10. In all cases, static analysis was used in design but P-delta effects were ignored in the calculation of the design story shear forces. For these two tall buildings, it is found that using a regular buckling-restrained braced frame did not lead to any improvement of the seismic stability of the structure compared to the Type MD CBF frame. Moreover, for both building heights, the regular BRB frames were found to have much lower L_{CC} values, close to 50%. Conventional CBFs with tension- and compression-acting braces typically exhibit additional story shear resistance after buckling of the braces has occurred, when the tension braces develop their yield strength (Tremblay, 2003). Although BRB members exhibit superior hysteretic behavior compared to conventional braces, BRB frames lack lateral overstrength once yielding has started at one level, which makes the system prone to story response and dynamic instability in the inelastic range. Using a lower

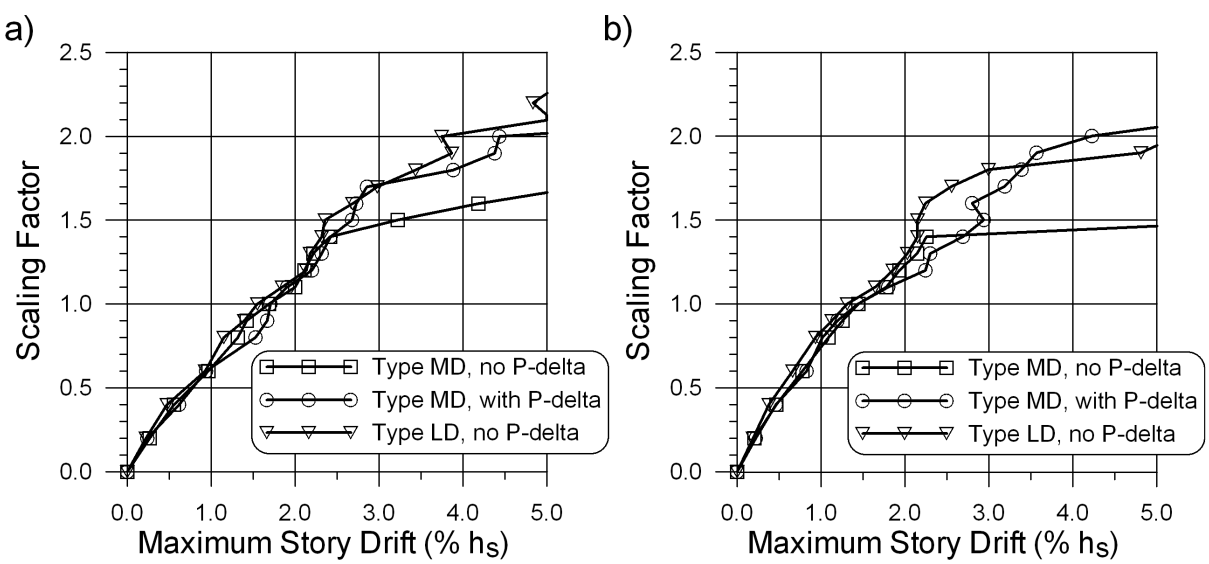

Fig. 8. Influence of the design story shear forces on the 50% percentile IDA curve for the conventional braced steel frames (static analysis): (a) 12-story frames; (b) 16-story frames.

R_d factor in design (for example, 3.0 instead of 4.0) would likely contribute to enhanced BRB IDA responses beyond those obtained for Type MD CBFs. BRB frames designed with such a higher capacity against dynamic instability would then probably represent a better solution compared to conventional CBFs in view of the lower demand imposed on the capacity protected elements and the reduced probability of brace fracture.

Figure 10 also shows that considerable benefit can be gained when buckling-restrained braces are combined with a mechanism capable of redistributing the inelastic demand over the building height. For the 12- and 16-story buildings studied herein, the dual BRB systems could withstand earthquake ground motions having 50% higher amplitude compared to the regular BRB frames, and L_{CC} values of 97% were obtained for these two dual BRB frames (Table 1). Interestingly, this significantly enhanced protection against dynamic instability was obtained with a limited amount of additional steel. It is believed that the behavior of this framing system will likely depend on the strength and stiffness of the members forming the elastic truss, and additional research is needed to tailor the design of the elastic truss system to reach the desired target performance level. In fact, the study presented in this paper formed the basis for subsequent work

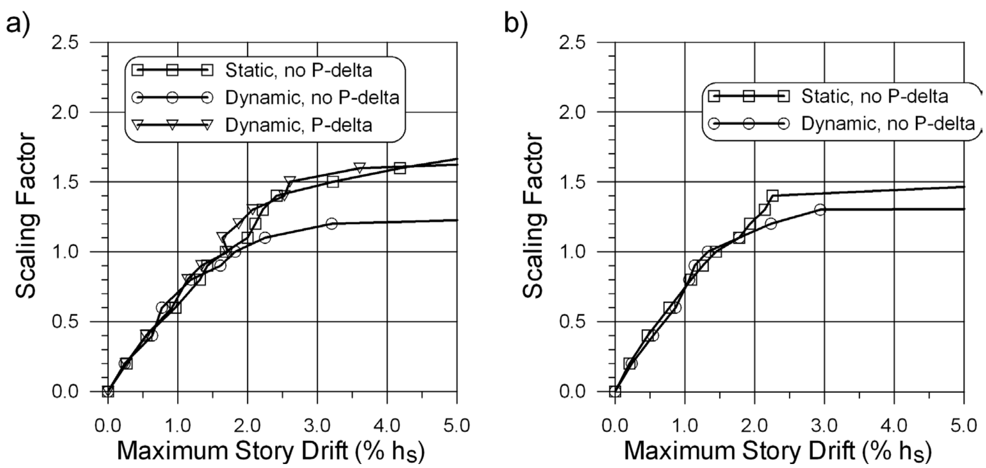

Fig. 9. Influence of the distribution of the lateral loads on the 50% percentile IDA curve for Type MD braced frames: (a) 12-story buildings; (b) 16-story buildings.

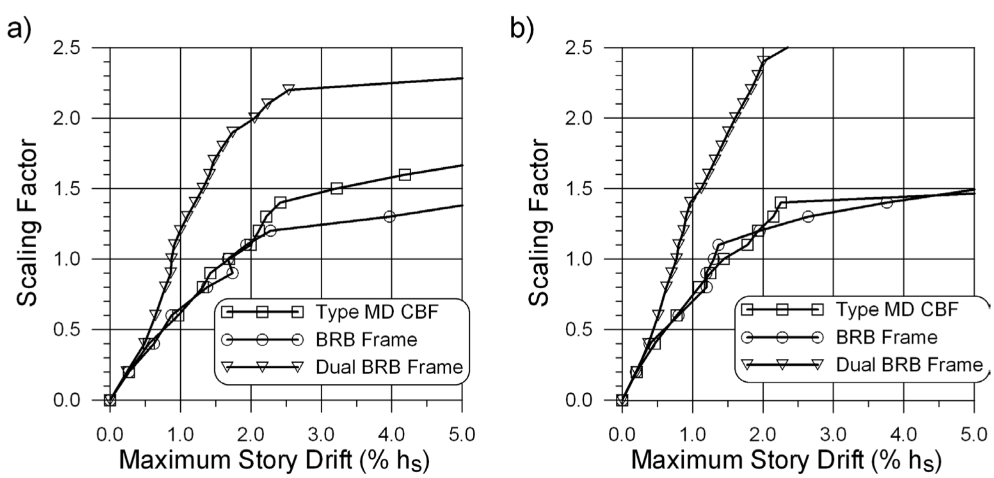

Fig. 10. Comparison of the 50% percentile IDA curves for Type MD CBFs, BRB frames, and dual BRB frames: (a) 12-story buildings; (b) 16-story buildings.

on the topic and more refined and effective design methods for the elastic truss members have been proposed recently (Merzouq and Tremblay, 2006).

CONCLUSION

An analytical study was carried out to evaluate the influence of several parameters on the seismic performance of multi-story braced steel frames including: the building height, the amplitude of the design story shears, the method of analysis, the type of bracing members, and the braced frame geometry. The structures were designed according to current Canadian code provisions and incremental nonlinear time history analyses were performed to assess the seismic performance of the structures. Brace fracture was not considered in the analysis. The main conclusions of the study can be summarized as follows:

- For conventional Type MD concentrically braced steel frames, the potential for dynamic instability due to story mechanism increases with the height of the building. For these structures, building height limits can therefore be imposed to ensure stable seismic response. In this study, it must be noted that the effects of building heights on frame stability was probably lessened to some extent due to the additional safety resulting from minimum design spectrum values specified in the Canadian building code for long period structures.

- Increasing the lateral resistance either by amplifying the design story shears to account for P-delta effects or by reducing the force modification factor led to a more robust response, likely because stronger structures respond more in the elastic range and, hence, are less prone to severe concentration of inelastic demand along their height.

- The use of a response spectrum analysis method to achieve a more uniform demand/supply ratio over the building height was not found to provide any benefit for seismic dynamic stability. This can be mainly attributed to the reduction in design earthquake forces that is permitted when the dynamic analysis method is used.

- Using buckling-restrained braces that exhibit more stable hysteretic response compared to conventional steel braces did not improve the stability of the taller braced frames. Coupling the buckling-restrained braces with a vertical truss system capable of mobilizing the energy dissipation capacity of the entire frame was shown to result in significantly enhanced seismic performance. Additional work is required to develop more refined design guidelines for such a promising dual system.

Further studies performed on a broader range of parameters and building samples are needed to confirm these conclusions. These additional studies should rely on more refined brace models capable of simulating more closely the actual hysteretic brace response including low-cycle fracture. Attention should also be devoted to the modeling of member joints that better reflect the actual strength and stiffness connection properties. Global dynamic stability response from analytical models should also be validated through physical testing.

ACKNOWLEDGMENTS

Financial support for this study was provided by the Natural Sciences and Engineering Research Council of Canada.

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