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SEISMIC PERFORMANCE OF MULTI-STOREY CONCENTRICALLY BRACED STEEL FRAMES DESIGNED ACCORDING TO THE 2005 CANADIAN SEISMIC PROVISIONS

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ABSTRACT

The seismic performance of multi-storey concentrically braced steel frames designed according to the latest seismic provisions of the CSA-S16 Standard is examined through nonlinear dynamic analysis. The buildings are located on a Site Class C in Victoria, BC. Their height varies from 2 to 16 storeys. The median estimates of the peak interstorey drift angles under the design level earthquake (2% in 50 years) range from 1.1 to 1.8%, which is less than the 2.5% applicable code limit. However, the computed values are between 17 and 80% higher than those predicted with the response spectrum analysis method. The interstorey drift angles and the roof drift angle to maximum interstorey drift angle ratios both increase with the building height. All buildings experienced residual deformations under the design level ground motions, with median residual interstorey drift angles varying between 0.2 and 0.9%. For all structures, the 50th percentile value of the peak ductility demand in the braces was generally less than the ductility level expected to cause brace fracture. Braces with lower slenderness ratios appear to be more prone to fracture. The confidence level against global collapse was verified for the 8-, 12-, and 16-storey buildings. All three structures were found to exhibit satisfactory protection against this failure mode.

Introduction

Past analytical studies have shown that inelastic response tends to concentrate in a few storeys along the height of multi-storey concentrically braced steel frames when subjected to strong ground shaking (Redwood et al. 1991; Martinelli et al. 2000; Tremblay 2000, 2003; Sabelli 2001; Tremblay and Poncet 2005). This behaviour is typically more pronounced in tall concentrically braced steel frames where inelastic demand and large storey drifts generally concentrate in the lower floors or in the upper levels. Such response can eventually lead to collapse by dynamic instability due to the significant P-delta forces that can be induced by the large storey drifts (Lacerte and Tremblay 2006). The concentration of storey deformations and brace ductility is mainly due to the degradation in storey shear that follows the buckling of the braces in compression.

Uriz and Mahin (2004), Dasgupta and Goel (2006) showed that this response can be accentuated if low-cycle fatigue fracture of the braces due to excessive cyclic inelastic demand is accounted for in the

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analysis. Concentration of storey drift demand as a consequence of brace fracture has been observed in past test programs (e.g., Foutch et al. 1986). Past test results also indicated that braces with low slenderness ratios are more prone to premature fracture (Tang and Goel 1987; Tremblay 2002; Tremblay et al. 2003). On this basis, it can be envisioned that stocky braces with low slenderness ratios used in tall buildings can represent a critical condition in terms of potential for brace fracture under seismic loading.

In the 2001 edition of CSA-S16 (CSA 2001), Type MD (Moderately ductile) concentrically braced frames with tension-compression acting braces were limited to buildings of 8 storeys or less to mitigate this behaviour. Type MD braced frames qualify for a ductility-related force modification factor of 3.0. This factor takes a value of 2.0 for Type LD (Limited-ductility) concentrically braced frames. The height limit for Type LD tension-compression braced frames was prescribed in CSA-S16-01 in view of the lower anticipated inelastic demand implied by the smaller force modification factor. In the 2005 edition of the National Building Code of Canada (NBCC) (NRCC 2005), the height limits for Types MD and LD braced frames were changed to 40 m and 60 m, respectively. This is taller than the 2001 restrictions assuming a storey height of 4 m. The 2005 supplement to CSA-S16-01 (CSA 2005) however requires that the design seismic forces be increased by 3% per meter of height for Type MD frames taller than 32 m. Similarly, the seismic design forces must be amplified by 2% per meter of height above 48 m for Type LD frames. The height limits and amplification in seismic loads are illustrated in Fig. 1. In the figure, the design seismic forces are relative to the value computed with a ductility-related force modification factor of 1.0.

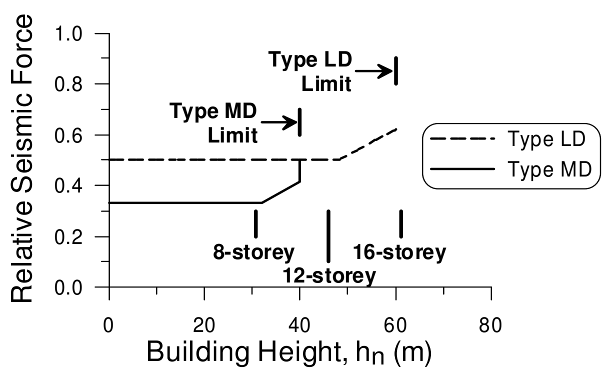

Figure 1. Relative minimum CBF design loads and height limits for CBF systems.

The analytical study presented in this paper was performed to assess the performance of tension-compression concentrically braced steel frames designed according to the recent seismic provisions included in CSA-S16S1-05. The response of structures having between 2 and 16 storeys in height is examined through nonlinear dynamic analysis performed under site specific seismic ground motion records. The response parameters of interest are the peak interstorey drift angles, the concentration of deformation demand along the structure height. The brace ductility demand is examined in relation with the potential for brace fracture. Residual deformations are also investigated. The level of confidence against global collapse instability is assessed through incremental dynamic analysis.

Building Design

Buildings Studied

A total of five buildings having 2-, 4-, 8-, 12-, and 16-storeys were selected for the study. The buildings were representative of typical office buildings. Table 1 gives the total building floor area, A, the building height, h_n, as well as properties of the structures that are used later for the seismic design. The building heights vary from 8.0 m to 61.2 m, thus covering well the possible range of application for Type MD and Type LD braced steel frames. Figure 2a shows the plan view of the 4-storey and taller buildings. As shown, these structures are symmetrical in plan and are laterally braced by two braced steel frames in each of the two orthogonal directions. The two-storey structure has the same structural arrangement except that larger overall plan dimensions are considered: 108.5 m x 108.5 m. This increase in size aimed at obtaining brace sizes with large cross-sections and low slenderness ratios, comparable to those selected for the taller structures. The elevation of the 8-storey bracing bent is illustrated in Fig. 2b. The same bracing configuration was used for all structures.

Table 1. Building Properties.

n	A (m^2)	h_n (m)	Type	R_d ( )	W (MN)	T_1 (s)	T_2 (s)	T_a (s)	V/W (%)	α_T ( )	U_2 ( )	Δ/h_s (%)
2	11770	8.0	MD	3.0	76	0.47	0.23	0.40	19.5	1.10	1.04	0.93
4	3660	15.6	MD	3.0	55	0.72	0.27	0.72	14.1	1.10	1.05	0.87
8	3660	30.8	MD	3.0	118	1.53	0.54	1.53	6.32	1.11	1.10	0.95
12	3660	46.0	LD	2.0	180	2.15	0.74	2.15	6.65	1.06	1.07	1.01
16	3660	61.2	LD	1.58	242	2.53	0.83	2.53	7.74	1.11	1.04	0.99

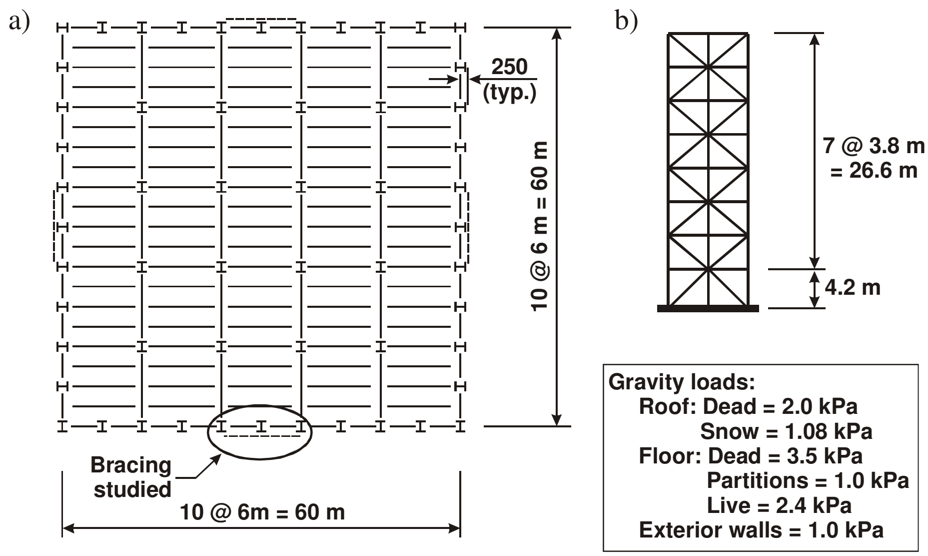

Figure 2. Buildings studied: a) Plan view (4-, 8-, and 12-storey buildings); b) Braced frame elevation (8-storey building shown).

Building Design

The buildings were assumed to be located on a Site Class C (soft rock) in Victoria, British Columbia, along the Pacific west coast. The design gravity loads are given in Fig. 2. The effects of earthquakes were determined using a three-dimensional response spectrum analysis of the structures. According to NBCC 2005 requirements, the member forces from the spectrum analysis performed in the direction of interest are adjusted such that the base shear from analysis is equal to $V_d = V_e I_E / R_o R_d$, except that the value of V_d cannot be less than 80% of the lateral earthquake design force, V, given by:

$$V = S(T_a)\, M_V\, I_E\, W / (R_d R_o) \qquad (1)$$

In these expressions, V_e is the elastic base shear from response spectrum analysis, I_E reflects the importance factor of the building, R_d and R_o are respectively the ductility- and overstrength-related force modifications factors, S is the design spectral response acceleration for the site, T_a is the building fundamental period used in design, M_V accounts for higher mode effects on base shear, and W is the seismic weight. In NBCC, the force obtained from Equation 1 need not exceed 2/3 the value of V determined with T_a = 0.2 s and must not be less than the value obtained with T_a = 2.0 s. These limits were considered in the calculations of V. For the site studied, the design spectrum ordinates, S, are equal to 1.20, 0.82, 0.38, 0.18, and 0.09 g for periods equal to 0.2, 0.5, 1.0, 2.0, and 4.0 s, respectively. Linear interpolation is used for intermediate period values. For braced steel frames, the design period is given by $T_a = 0.025\ h_n$. Alternatively, the period from modal analysis can be used provided that the so-computed period does not exceed $T_a = 0.05\ h_n$. In NBCC, M_V = 1.0 for braced steel frames located along the Pacific west coast. The buildings were of the normal importance category with I_E = 1.0. The seismic weight includes the dead load and 25% of the roof snow load. The values of W are given in Table 1.

As permitted in CSA-S16, Type MD braced frames with an R_d factor of 3.0 were used in the 2-, 4-, and 8-storey buildings. The height of the 12-storey building exceeded the limit for Type MD braced frames (46 m > 40 m) and a Type LD system with an R_d factor of 2.0 was adopted for that structure. For the 16-storey building, the height just exceeded the 60 m limit for Type LD frames. Nevertheless, the structure was designed as a Type LD frame to assess the appropriateness of CSA-S16 seismic provisions for structures approaching the height limit specified for this system category. However, the seismic loads had to be amplified by 26.4% since the height exceeded 48 m: 26.4% = 2% x (61.2 m - 48 m). This increase in seismic loads resulted in an equivalent R_d factor of 1.58. Table 1 summarizes the braced frames types and the equivalent R_d factors used in design. The relative design seismic loads are also illustrated in Fig. 1 for each of the 5 buildings. For both Type MD and LD systems, the factor R_o = 1.3.

In the design process, the fundamental period from analysis, T_1, was determined at each iteration cycle and the smaller of T_1 and the upper limit $T_a = 0.05\ h_n$ was retained to calculate V. The final values of the periods in the first two modes of vibration, T_1 and T_2, and the value of T_a are given in Table 1. As shown, the upper limit $T_a = 0.05\ h_n$ governed only for the 2-storey building. For all structures, the base shear V_d from the analysis exceeded 0.8 V and no adjustment was needed. The resulting V_d/W values are also given in Table 1. Torsional effects due to accidental eccentricity and P-delta effects were also included in the analysis. In Table 1, the factor α_T corresponds to the increase in storey shear at the base of the bracing bent studied due to torsion, as obtained by comparing the results from 3D analyses performed with in-plane rotation of the building released and restrained, respectively. The CSA-S16 P-delta amplification factor U_2 at the base of the structure is also given in Table 1. The table also gives the maximum interstorey drift angle value over the building height, Δ/h_s, as predicted by response spectrum analysis. All computed values are less than the limit of 2.5% prescribed in NBCC for buildings of the normal importance category.

The bracing members were designed for compression assuming an effective length factor of 0.9. Square HSS structural tubing conforming to ASTM A500, gr. C (F_y = 345 MPa) were used for all braces except at the first floor of the 2-storey building and at levels 1 to 7 and level 9 of the 16-storey building where W-shapes made of ASTM A992 steel with F_y = 345 MPa were selected. W shapes with the same material were also used for the beams and columns. According to CSA-S16 provisions, beams and columns were designed for gravity load effects in combination with axial loads arising from buckling and yielding of the braces. Column segments spanning over two consecutive floors were considered. The cross-section dimensions of the HSS braces ranges between 152 mm and 305 mm, whereas W360 sections were used for the W shaped braces. The brace slenderness ratios computed with the assumed K value of 0.9 varies between 55 and 116 with an average value of 69. All braces meet the CSA-S16 seismic limits on width-to-thickness ratios. The brace gusset plates were designed with the free distance allowing ductile rotational behaviour of the gussets upon brace out-of-plane buckling. Once the design was completed, it was found that the effective length factor for the braces including connection size and rotational restraint effects was typically equal to 0.75, instead of the 0.9 value assumed in design. This resulted in brace slenderness ratios varying between 46 and 97 with a mean value of 58.

Modelling and Ground Motions

Nonlinear dynamic analysis of the structures was performed with the OpenSees computer program (McKenna and Fenves 2004). The numerical models included the bracing bent studied as well as all the columns in the buildings that are laterally stabilized by the braced frame analyzed. Rigid diaphragm response was assumed at all floors and torsional response of the building was taken into account indirectly as discussed later. Hence, for simplicity, all the members of the model could be re-arranged in a 2D representation. Three-dimensional analysis was however performed to reproduce the out-of-plane buckling response of the bracing members. These members were modelled using non-linear beam-column elements with a fibre discretization of the cross-section. Each brace was represented with 8 elements having 4 integration points, and a co-rotational formulation was used to include geometric nonlinearities. The uniaxial Giuffré-Menegotto-Pinto (STEEL02) steel material model with kinematic and isotropic hardening was assigned to the brace fibres to simulate Bauschinger effect under cyclic inelastic loading. An expected yield strength value (R_yF_y) of 380 MPa was specified for the steel. Rigid beam elements were used between the beams and columns and the brace ends to simulate the size of the gusset plates. The braces were connected to these rigid elements by means of zero-length rotational springs exhibiting inelastic flexural response to reproduce the out-of-plane bending stiffness and hysteretic behaviour of the gusset plates upon brace buckling. Further details and validation of this modelling technique can be found in Aguerro et al. (2006).

The beams and columns were modelled using elastic beam elements with fibre plastic hinges concentrated at the member ends (Beam with Hinges elements). The length of the plastic hinges was set equal to the dept of the members. The beams in the braced frame were assumed to be rigidly connected to the columns at brace connection locations. Other beam-to-column connections were assigned a flexural strength equal to 10% of the beam flexural capacity. The columns of the bracing bents were modelled with fixed base connections and full flexural continuity at splice connections. The gravity columns were assumed to be pinned at their bases and their splices were assigned to have 10% of the flexural strength of the smaller connected members.

Gravity loads corresponding to the dead load plus 50% of the live load and 25% of the roof snow load was applied to the structure. The structures were subjected to an ensemble of 10 seismic records from past earthquakes and 10 simulated ground motion time histories. The site conditions and magnitude-distance characteristics of these signals were representative of the conditions at the site. The motions were scaled to match on average the design spectrum, as illustrated in Figure 3. For consistency with design assumptions, the ground motions were then multiplied for each building by the torsion amplification factor, α_T, given in Table 1. The Newmark-Beta integration technique was used for the transient analyses with a time step of 0.005 s. Rayleigh damping with 3% of critical damping in the first two modes was considered.

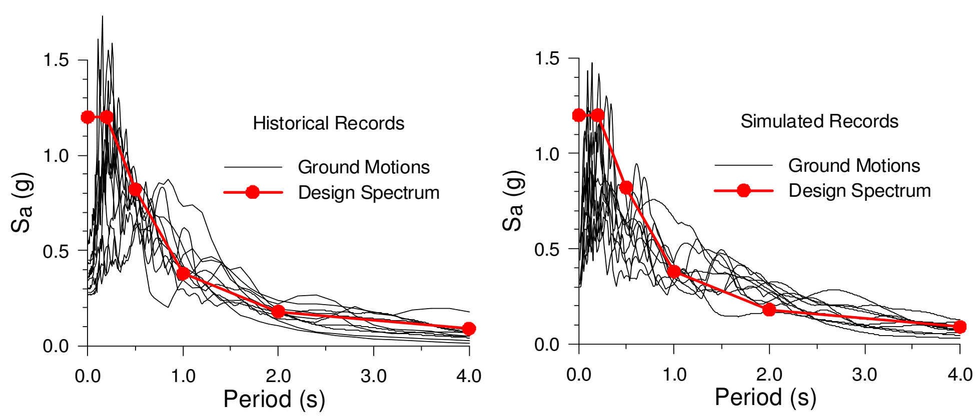

Figure 3. 5% Damped acceleration response spectra of the scaled ground motion time histories.

Seismic Performance

Response at the Design Earthquake Level

The peak values at every floor of the interstorey drift angle (Δ/h_s) and residual interstorey drift angle (Δ_r/h_s) were determined for each scaled ground motion. The maximum peak values along the height of each building were kept and the statistics for the entire ground motion ensemble are given in Table 2. In view of the large number of earthquake records considered (20), median (50^{th} percentile) response estimates are used herein to evaluate the performance of the structures, as recommended in recent code and standard documents (e.g., ASCE 2005). Other statistical values (84^{th} percentile and maximum values) are presented to assess the variability in the response parameters. The 50^{th} and 84^{th} estimates of Δ and Δ_r are graphically presented in Figs. 4a&b, respectively. The interstorey drift angle values predicted from response spectrum analysis and given in Table 1 are plotted in Fig. 4a for comparison purposes.

Table 2. Interstorey Drift Demand.

n	Δ/h_s (%)			Δ_r/h_s (%)			DCF ( )				
	50^{th}	84^{th}	max	50^{th}	84^{th}	max	50^{th}	84^{th}	max	Ref. 1	Ref. 2
2	1.09	1.50	3.51	0.16	0.35	0.68	1.37	1.58	1.64	1.80	1.78
4	1.08	1.64	3.42	0.17	0.45	0.82	1.70	1.94	2.03	1.82	1.80
8	1.40	2.17	4.24	0.92	1.50	3.45	1.91	2.06	2.22	1.84	1.82
12	1.59	2.48	4.62	0.32	0.60	3.29	2.40	2.71	2.89	1.86	1.80
16	1.78	2.20	2.70	0.30	0.59	1.08	2.47	2.99	5.62	1.99	1.81

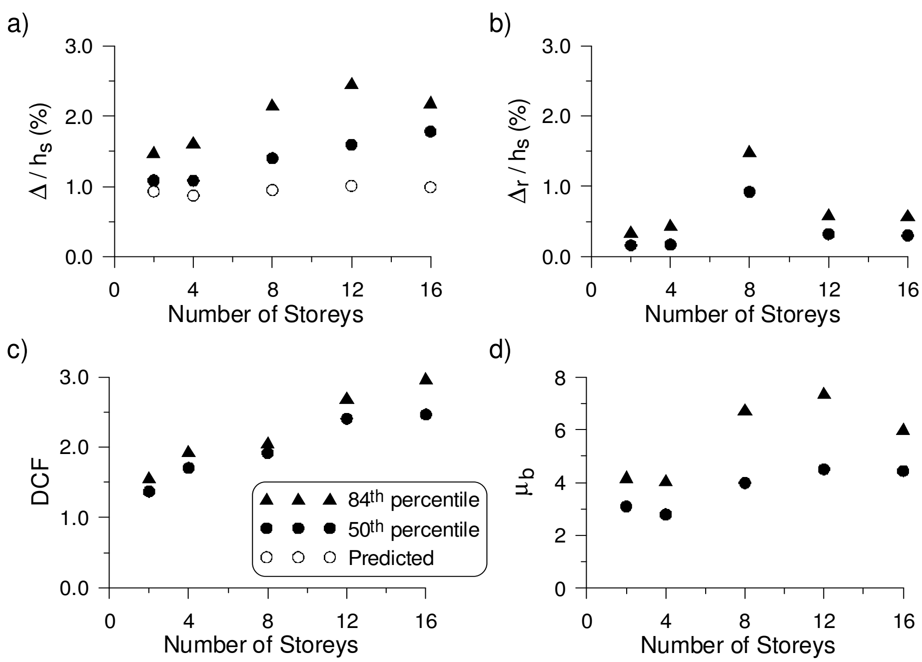

Figure 4. Statistics of maximum peak response parameters: a) Interstorey drift angle; b) Residual interstorey drift angle; c) Drift concentration factor (DCF); and d) Brace ductility demand.

As shown, the median estimates of the peak interstorey drift angle steadily increase with the building height. All values are less than the 2.5% limit prescribed in NBCC 2005 but the demand from the ground motions exceeds the value predicted by the response spectrum analysis in design for all buildings. The ratios between the 50^{th} drift values and the spectrum analysis predictions vary between 1.17 for the 2-storey frame to 1.80 for the 16-storey buildings. As anticipated, all structures experienced inelastic deformations through brace buckling and yielding, which led to residual (permanent) deformations at the end of the earthquake records. Figure 4b shows that the median residual storey inclinations range from 0.16% to 0.92%. NBCC 2005 does not provide acceptance criteria regarding this response parameter. However, structures with residual out-of-plumbness larger than 0.2% (1/500) will need post-earthquake structural evaluation as this value represents the maximum erection tolerance specified in CSA-S16.

The drift concentration factor (DCF) corresponds to the ratio of the maximum peak interstorey drift angle along the building height to the peak roof displacement divided by the building height. This parameter is used as an indicator of the distribution of the demand over the building height: a value 1.0 for DCF indicates uniform storey deflections at peak displacement during an earthquake while DCF greater than 1.0 reflect concentrations of the demand along the height of a structure. As shown in Table 2 and Fig. 4c, the nonlinear dynamic analysis results clearly revealed an increase in damage concentration as the building height is increased. Expressions have been proposed in past studies to predict DCF values for multi-storey building structures. MacRae et al. (2004) suggested that the DCF in braced steel frames varies as a function of the number of storeys, the vertical distribution of the lateral resistance of the seismic force resisting system, the flexural stiffness of the columns at the first level relative to the bracing bent lateral stiffness, and the expected global ductility level. Values obtained with this model are given in Table 2 (Ref. 1). Another model by Miranda (1999) is based on the global ductility level, the number of levels, and the governing deformation mode of the structure (shear *vs* flexure). Values computed with this second model are also reported in Table 2 (Ref. 2), assuming an intermediate deformation mode for braced steel frames (factor β_2 = 1.6). In both models, the ductility level was taken equal to the R_d factor used in design. As shown, the predictions are fine for the 2- to 8-storey buildings, but concentration of deformation higher than anticipated was experienced by the 12- and 16-storey structures, even if these structures had been designed with amplified seismic loads to mitigate this effect.

The maximum value of the peak brace ductility demand along the structure height is shown in Fig. 4d. The ductility level in the braces was determined based on the net brace length between the hinge lines in the gusset plates and using a yield strain based on the expected material properties. As anticipated, the variation of the brace ductility with the building height is similar to those observed for the peak interstorey drift angle and DCF parameters. In order to evaluate the potential for brace fracture due to possible excessive inelastic deformations, the brace demand is compared to the empirical expression proposed by Tremblay (2002) for the brace ductility at fracture, μ_f:

$$\mu_f = 2.4 + 8.3\,\lambda \qquad (2)$$

This ductility limit is a function of the brace dimensionless slenderness parameter, $\lambda = (F_y/F_e)^{0.5}$, where F_e is the brace elastic buckling stress. The ductility μ_f corresponds to the sum of the peak ductility reached in compression, μ_c, and tension, μ_t, prior to fracture of a brace subjected to cyclic inelastic loading. Equation 2 was obtained from past experimental data. It represents the best linear fit of the test data with a COV of 0.25 for the test-to-predicted ratios. The mean value and the mean ± one standard deviation value (M±STD) of μ_f are plotted in Fig. 5. The 50^{th} and 84^{th} percentile values of the peak ductility demand for all braces of all five buildings from nonlinear dynamic analysis are plotted in Figs. 5a and 5b, respectively. In the figure, it is assumed that the braces experienced the same ductility level in tension and compression, i.e. $\mu_t + \mu_c$ in the figure corresponds to two times the computed peak ductility values. The brace effective length factor K = 0.75, which more closely represents the as-designed end conditions, was used in the calculation of the brace slenderness parameter λ. As shown, the braces generally exhibit satisfactory performance against fracture based on median estimates (Fig. 5a). The most critical braces appear to be those which have a slenderness parameter λ in the vicinity of 0.75 (KL/r ≈ 55) and are used in the 12- and 16-storey buildings. Relatively lower demand was imposed on the stockiest braces ($\lambda \approx 0.6$, KL/r ≈ 45) as well as on the more slender braces ($\lambda > 0.9$, KL/r > 65), which suggests that these braces could be less prone to fracture. A similar trend is observed when examining the 84th percentile demand values in Fig. 5b.

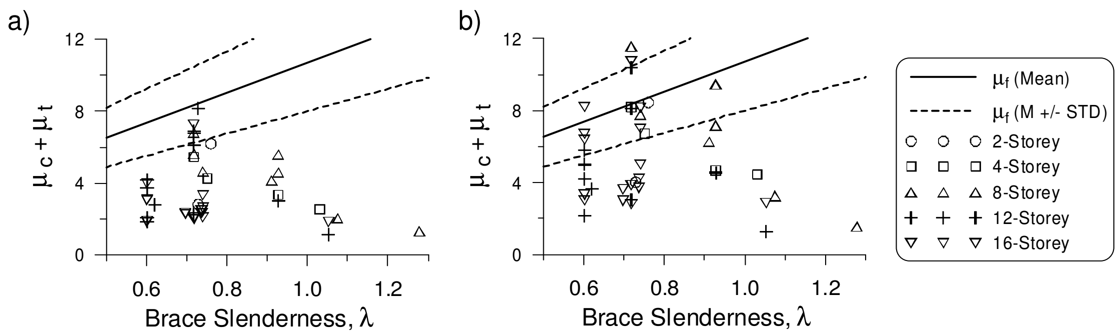

Figure 5. Peak ductility demand in the braces (assuming symmetrical response) vs anticipated brace ductility at fracture: a) 50th percentile demand values; b) 84th percentile demand values.

Incremental Dynamic Analysis

Incremental dynamic analysis was performed on the 8-, 12-, and 16-storey buildings to assess their level of confidence against global collapse. The analysis was carried out for the 10 historical ground motion records. Figure 6 shows the 50th and 84th percentiles of the maximum peak interstorey drift angles along the height of the buildings upon incrementing the amplitude of the ground motion records. In the figure, a scaling factor, S_F, equal to 1.0 corresponds to the 2% in 50 year design hazard level. All three structures displayed a very robust response. The 8-storey structure shows a gradually softening behaviour whereas the 12- and 16-storey structures exhibited a stiffening response for S_F between approximately 1.0 and 3.0.

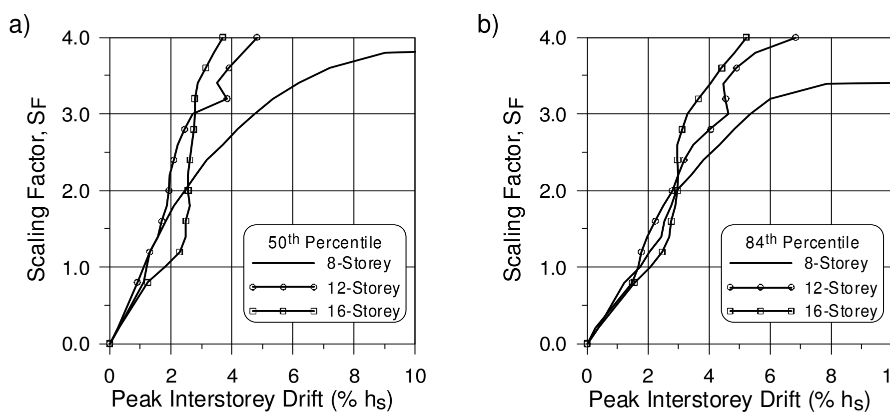

Figure 6. Peak interstorey drifts under incremental dynamic analysis: a) 50th percentile values; b) 84th percentile values.

The level of confidence against global collapse was determined following the procedure proposed in FEMA 350 (FEMA 2000) and described in Tremblay and Poncet (2005). For the three structures, that level of confidence was found equal to 99.9%, well above the minimum value of 90% recommended in the FEMA 350 document. Tremblay and Poncet (2007) found unsatisfactory performance for 12-storey and 16-storey concentrically braced steel frames designed with an R_d factor of 3.0, when the amplification of the design seismic loads to account for the building height and P-delta effects were not included in design.

In that same study, the 12-storey structure was also found to exhibit inadequate performance against global collapse when P-delta effects alone were considered in design. The results presented in this paper indicate that the new requirements of CSA-S16S1-05 for amplified seismic loads for tall concentrically braced steel frames can lead to stable inelastic seismic response.

Conclusions

Nonlinear dynamic analysis was carried out on 2-, 4-, 8-, 12-, and 16-storey concentrically braced steel frames to verify the adequacy of design provisions that were recently introduced in CSA-S16S1-05 to improve the seismic performance of tall concentrically braced steel frames. Under the design ground motion level corresponding to a probability of exceedance of 2% in 50 years, all five structures experienced median maximum peak interstorey drift angle values that meet the 2.5% limit prescribed in NBCC 2005 for buildings of the normal importance category. The median peak interstorey drift angles generally increased with the building height, varying from 1.1% for the lowest structures to 1.8% for the tallest frame. For the 8-storey and taller structures, these deformations exceeded by 50 to 80% the values obtained from response spectrum analysis. The study also revealed that the 12- and 16-storey structures exhibited significant concentration of storey deformations along their height, in excess of the values predicted using models published in the literature. However, such larger drift ratios and deviations from uniformly deformed shapes were not critical for the overall stability of the taller structures as the 8-, 12-, and 16-storey braced frames demonstrated a very robust response against global collapse when subjected to incremental dynamic analysis. In all three cases, the level of confidence against collapse was equal to 99.9%, which is well beyond the recommended minimum value of 90%. These results seem to validate the adequacy of the new seismic requirements of CSA-S16S1-05 for tall braced steel frames.

The study indicated that braces with a low slenderness ratio ($KL/r \approx 55$) would be the most critical for fracture due to low-cycle fatigue, especially when used in taller braced frames. A lower inelastic demand was observed for more slender braces ($KL/r > 65$) as well as for braces with very low slenderness ratios ($KL/r \approx 45$). The results also showed that braced steel frames can experience permanent storey deformations under design level earthquakes. This may lead to high repair costs or, in some cases, to the demolition of the structure. Both the effects of residual deformations and the assessment of the potential for brace fracture need to be examined further in future studies. It is also recommended that the findings of this study be validated for other braced steel frame configurations.

Acknowledgments

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