SEISMIC DESIGN AND BEHAVIOUR OF CHEVRON STEEL BRACED FRAMES

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SUMMARY

This paper examines four different seismic design approaches for multi-storey chevron steel braced frames. The first method is in accordance with current Canadian design provisions. In the other methods, the seismic loads are reduced but the beams are strengthened to sustain various levels of post-buckling brace loading conditions. The study indicates that current design procedures lead to a less expensive design but result in a behaviour that is less desirable. Building height limitations are proposed which are applicable to each design procedure.

INTRODUCTION

Despite their poor seismic performance, the use of chevron (inverted-V) braced frames still represents a very popular means of resisting lateral loads in steel building structures. Under severe earthquake ground motions, the braces are expected to buckle and loose their compressive strength. The beams are then pulled downward due to the combined action of the gravity loading and the tension acting braces (Fig. 1a). Unless the beams are designed to carry this net vertical load together with the axial loads that develop from the braces, a plastic hinge eventually forms at mid-span of the beams before the tension braces reach their yield tensile capacity (Fig. 1b). This behaviour results in a severely pinched hysteretic lateral response with strength and stiffness deterioration, which can lead to the formation of soft storeys in multi-storey frames. In Canada, such chevron braced frames are classified under the Braced Frames with Nominal Ductility category (NDBF), as defined in the CSA-S16.1 Standard [CSA, 1994]. The prescribed seismic loads for the NDBF category are 50% higher than those specified for the Ductile Braced Frame (DBF) category.

Past research [e.g. Khatib et al., 1988; Remennikov and Walpole, 1998] has shown that the seismic performance of chevron braced frames can be improved if stronger beams are used. Recent U.S. code provisions [AISC, 1997; ICBO, 1997] now permit the inclusion of chevron braced frames in the Special Concentrically Brace Frame category, provided that the beams are designed to sustain the net vertical force that develops upon yielding of the tension braces after buckling of the compression braces has occurred. Remennikov and Walpole suggest that this requirement could be relaxed, without detrimental effects, by considering only a fraction of the yield tensile strength of the braces in the design of the beams.

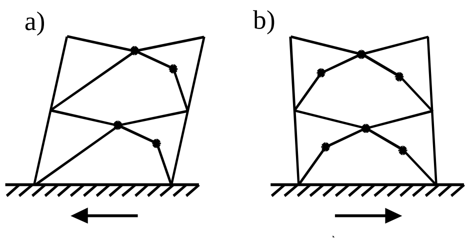

Figure 1: Inelastic response of chevron braced frames

In this paper, the economical impact and the seismic performance expected from various design approaches are examined and the possible inclusion of chevron braced frames in the CSA-S16.1 DBF category is investigated.

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The performance of chevron braced frames of the NDBF category designed according to current provisions is also evaluated. Typical braced frames are designed according to current and proposed design provisions and their behaviour is studied through nonlinear dynamic analysis. The frames are representative of Canadian practice, i.e., simple shear beam-to-column connections are used and, hence, all lateral loads are assumed to be resisted by the truss action provided by the vertical bracing bents. This system has a very limited ability to redistribute in the vertical direction the inelastic demand and, therefore, is prone to soft storey formation and dynamic instability, especially if the braced frame develops a poor storey shear-storey deformation hysteretic response. In this study, special consideration has been given to this behaviour by submitting the structures to two levels of ground motion.

DESCRIPTION AND DESIGN OF THE STRUCTURES

Geometry and Loading

The structure of an office building located in Vancouver, B.C., was studied. The floor plan is shown in Fig. 2. Four different heights were considered: 2, 4, 8, and 12 storeys. The storey height at every floor is 3.8 m. For all buildings, the two identical bracing bents resisting seismic loads in the north-south direction were studied. The building structures were designed according to the 1995 National Building Code of Canada (NBCC) [NRCC, 1995]. The following gravity loads were considered in the calculations: a roof dead load of 1.2 kPa (2- and 4-storey buildings) and 3.7 kPa (8- and 12-storey buildings), a floor dead load of 4.5 kPa, a roof snow load of 1.48 kPa, and an occupancy floor live load of 2.4 kPa. The weight of the exterior cladding was taken equal to 1.2 kPa.

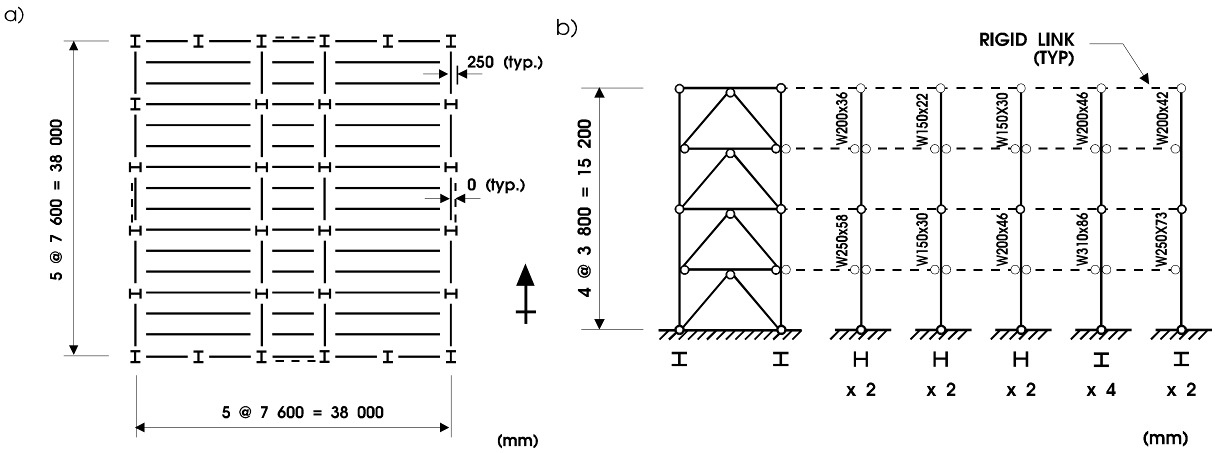

Figure 2: a) Plan view of the buildings; b) Analytical model of the 4-storey structure

The braced frames were designed for a seismic horizontal load V = vSFIW (U/R), where v is the velocity ratio for the site (0.21 for Vancouver), S is the seismic response factor, F is the foundation factor, I is the importance factor, W is the seismic weight of the structure, U is a calibration factor (U = 0.6), and R is the force modification factor. The value of S varies with the fundamental period of the structure, T, and the seismic zone at the site. For Vancouver, the variation of Sa (the product of v and S) with T is shown in Fig. 5. The fundamental periods prescribed in the NBCC were equal to 0.25 s, 0.49 s, 0.98 s and 1.47 s for the 2-, 4-, 8-, and 12-storey buildings, respectively. For all structures, the periods computed from free vibration analysis exceeded these values and the S factors could then be reduced to 80% of their initial values, as permitted in the NBCC.

The importance factor and the foundation factor were taken equal to 1.0 (structure of normal importance on stiff soil). The seismic weight, W, included the floor and roof dead loads, the weight of the exterior walls, and 25% of the roof snow load. It was equal to 10050 kN, 24790 kN, 57540 kN, and 87030 kN for the 2-, 4-, 8-, and 12-storey buildings, respectively. As discussed later, the R factor was assigned a value of 2.0 or 3.0 depending upon the design approach. The seismic load V was then distributed over the height of the structures according to the NBCC static procedure. A concentrated lateral force equal to 6.9 % and 10.4 % of the base shear had to be applied at the top of the 8- and 12-storey buildings, respectively. In-plane torsion was neglected and each frame in the N-S direction was therefore designed for half the total applied lateral loads. P-delta effects were also ignored in the design and prescribed drift limitations did not govern the member selection.

Design

Four different design methods were considered for the bracing bents. The first approach follows the current S16.1 provisions for braced frames with nominal ductility (NDBF). For these frames, an R factor of 2.0 is specified in the NBCC for the determination of the seismic loads and there is no special provisions for which the beams must resist the brace loads that develop during an earthquake after buckling of the braces. In the other three design approaches, the frames are assumed to classify under the DBF category with an R factor of 3.0 and the beams are sized to have minimum resistance against the post-buckling brace loads.

In the NDBF frames, the bracing members are the first members to be selected. They are sized to carry the combined gravity and seismic induced forces (D + 0.5L + E) assuming the frame remains elastic. All braces were cold formed square HSS members made of G40.21-350W steel (Fy = 350 MPa) and an effective length factor, K, equal to 0.8 was used in design to account for the size and fixity of their end connections. In order to exhibit ductile behaviour, S16.1 also requires that the brace slenderness ratio, KL/r, be less than 102 and that the width-to-thickness ratio of their cross section, b/t, does not exceed 17.6. These additional provisions governed the design at the uppermost floor level in all frames. The slenderness ratio of the selected braces varied between 55 and 92.

Beams and columns are sized to carry the factored gravity loads alone (1.25D + 1.50L) assuming an elastic response. Under seismic ground motion, they also have to remain elastic until buckling of the braces occurs. Thereafter, flexural yielding may develop in the beams at the brace intersection points. Elastic behaviour prior to brace buckling is checked by applying the gravity loads (D + 0.5L) together with the lateral loads that are required to reach the buckling resistance of the compression braces at every floor. In this calculation, the beams are considered as beam-columns supported at their mid-span by the braces connected below. The S16.1 requires that the beams must also be checked for the post-earthquake condition, i.e., the beams being capable of supporting their tributary gravity loads (D + 0.5L) while neglecting the support provided by the buckled braces. The selected beams were Class 1 (plastic design) W shapes made of G40.21-350W or ASTM-A572, gr. 50 steel, depending upon the shape availability. Two-storey tier columns were assumed for which W and WWF shapes made of G40.21-350W steel were chosen.

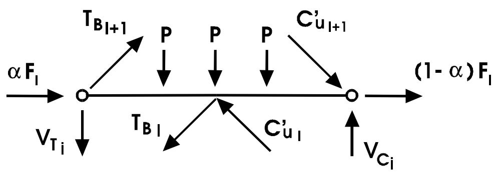

Figure 3: Brace forces and gravity loading acting on beams in DBF structures

The same design approach was used for the DBF frames except that an additional load case was considered for the beams and the columns. In the DBF-100 design, the beams are also checked as simply supported beam-columns carrying the gravity loads (D + 0.5L) together with the brace loads assuming the compressive brace develops its post-buckling resistance, C'u, and that the tension brace reaches its tensile resistance, AgFy. In this study, the post-buckling resistance, C'u, was taken equal to 0.2 AgFy, which is representative for braces with intermediate slenderness ratios [Nakashima et al., 1992]. At every floor, the axial compression and tension forces acting in the beam, in combination with the vertical unbalanced brace load, are obtained by studying the horizontal equilibrium of the beam (Fig. 3). In this figure, T_{Bi} and T_{Bi+1} are the yield load of the braces at and above the storey under consideration. The coefficient α accounts for the location of the bracing bent along the frame in the structure. For the buildings studied, α is taken as 0.5. The columns are checked for the axial loads induced by the forces V_{Ti} and V_{Ci} at each level.

For the DBF-80 and DBF-60, only 80% and 60%, respectively, of the brace yield load is considered for the tension brace connected under the beam (T_{Bi}). The 80% value was proposed by Remennikov and Walpole and the 60% value has been considered to examine the effects of relaxing further the beam design loads. In the computation of beam axial loads, the force T_{Bi+1} is the maximum brace force that can develop assuming a complete mechanism has formed in the storeys above the level under consideration (Fig. 4). This force is based on the actual strength of the beams selected at these levels. Columns are also checked for the vertical forces introduced at every floor when the mechanism shown in Fig. 4 has developed over the full height of the frame.

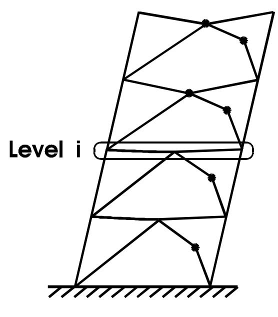

Figure 4: Partial mechanism studied for the design of beams

Table 1 gives the design base shear ratio, V/W, the tonnage of steel per bracing bent, and the computed fundamental period for each of the 16 braced frames designed in this study. The numbers in brackets correspond to the ratio of the steel weight required for the DBF systems relative to the NDBF design. As shown, the benefits of reducing the seismic loads for the DBF frames is always significantly offset by the increase in material for the beams. The difference decreases, however, as the building height is increased and when the design tension brace load is reduced.

Table 1: Properties of the bracing bents studied

Number of Storeys		NDBF	DBF-100	DBF-80	DBF-60
2	V/W	0.151	0.101	0.101	0.101
	Steel tonnage (t)	1.77	3.43 (1.93)	3.21 (1.81)	2.67 (1.50)
	Fundamental period (s)	0.45	0.46	0.46	0.47
4	V/W	0.106	0.071	0.071	0.071
	Steel tonnage (t)	4.82	8.19 (1.70)	7.58 (1.57)	6.44 (1.34)
	Fundamental period (s)	0.89	0.89	0.90	0.91
8	V/W	0.076	0.050	0.050	0.050
	Steel tonnage (t)	14.6	21.5 (1.47)	19.7 (1.35)	17.7 (1.21)
	Fundamental period (s)	2.02	2.06	2.09	2.11
12	V/W	0.062	0.041	0.041	0.041
	Steel tonnage (t)	29.8	46.7 (1.57)	35.9 (1.20)	32.7 (1.10)
	Fundamental period (s)	3.17	3.29	3.37	3.39

ANALYTICAL MODEL AND EARTHQUAKE RECORDS

The seismic performance of the braced frames was evaluated through nonlinear time history dynamic analyses performed using the Drain-2D computer program (Powell and Kanaan, 1973). The model (Fig. 2b) included the bracing bent that was studied, as well as the gravity columns that are stabilised by this bracing bent. The bracing members were modelled using the inelastic brace buckling element with pinned ends developed by Jain and Goel (1978). The post-buckling resistance of the braces, C'u , was also set equal to 0.2 AgFy. Beam-column elements with plastic hinges forming at their ends were used for the beams and columns. A bi-linear hysteretic response with 2% strain hardening was assumed for these elements. All beams were assumed to be pin-connected to the columns and all columns were made continuous over two storeys with zero-moment connection splices. A Newmark constant acceleration integration scheme with a time step of 0.0005 s was used throughout the study. P-Δ effects were considered in the calculations with 100% of the dead load and 50% of the live load applied to the structure. Rayleigh damping equal to 5% of critical damping in the first two modes was adopted in all analyses.

Table 2: Ground motion records and scaling factors for A and B ensembles

Event	Station	Comp.	PHA (g)	PHV (m/s)	Scaling Factor	
					A	B
1940 Imperial Valley, Ca	El Centro	S00E	0.35	0.33	0.63	0.8
1971 San Fernando, Ca	Hollywood Storage, L.A.	N90E	0.21	0.21	1.0	1.3
1971 San Fernando, Ca	Hollywood Storage, L.A.	S00W	0.17	0.17	1.25	1.5
1949 Western Washington, Wa	Olympia, Highway Test Lab	N04W	0.16	0.21	1.02	1.3
1983 Coalinga aftershock, Ca	Oil Fields Fire Station	N270	0.22	0.16	1.31	1.3
Simulated motion, $M_w = 7.2$	R = 70 km	-	0.11	0.19	1.11	1.7

The structures were subjected to two ensembles of ground motion time histories. Both ensembles contained the same six records described in Table 2. Five ground motions were produced by earthquakes that occurred along the western coast of North America and one record is a simulated time history generated by Atkinson and Beresnev [1998]. Ensembles A and B differed only in the scaling factors that were used. The records of ensemble A were scaled to match the design peak ground velocity at the site (0.21 m/s). Figure 5 compares the computed mean and mean plus one standard deviation (mean+SD) spectra for this ensemble to the NBCC design spectrum (v times S). As shown, the mean+SD spectrum corresponds well with the design spectrum, which suggests that the ground motion level in ensemble A is compatible with the level used in design. Recent earthquakes have revealed, however, that measured ground motion amplitudes can significantly exceed the demand predicted using a probabilistic seismicity model. Therefore, the B ensemble in this study was used to assess the robustness of the structures when subjected to such stronger ground motions. In this ensemble, each record was scaled to match, in average, the design spectrum.

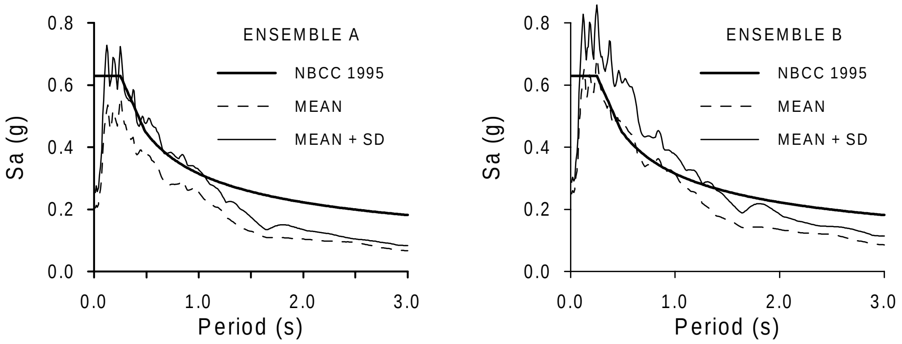

Figure 5: Design spectrum and computed 5% damping acceleration response spectra for ground motion ensembles A and B

RESULTS

Four parameters are examined to evaluate the seismic performance of the bracing bents: the storey drift, the tension force in the braces, the ductility experienced by the compression braces, and the bending moment in the columns of the bracing bent. The storey drift is normalised with respect to the storey height, hs. The brace tension load and the column moments are normalised to the yield resistance of the brace, AgFy, and the plastic moment of the columns, respectively. The compression brace ductility is obtained by dividing the brace axial shortening by the brace deformation at yield (9.40 mm). For each parameter, the maximum peak values over the building height reached during each ground motion record was retained. For both ensembles, the mean plus one standard deviation of these maximum values is presented in Fig. 6 for all 16 bracing bents studied.

Figure 6a shows the computed maximum storey drifts. This parameter is a good indicator of both the structural and non-structural damage experienced by buildings. For instance, it is suggested [SEAOC, 1996] that a building is operational with light damage if the storey drift is less than 0.005 hs and that structural damage is expected to develop at a storey drift of 0.015 hs or higher. In the NBCC 1995, the limit on the inelastic storey drift is 0.02 hs. In the analyses, dynamic instability or situations where the structures were on the verge of collapse (storey drift exceeding 0.10 hs) were also observed. The number of such events is given in Table 3 for each combination of bracing bent and ground motion ensemble. The response parameters for these cases were not included in the computation of the statistical results presented in Fig. 6. Hence, the results shown in Fig. 6 for the bracing bent-ground motion ensemble combinations for which instability was observed only reflect the structural behaviour under the remaining, less critical ground motions which the building could resist.

Table 3: Occurrences of dynamic instability under A and B ground motion ensembles

Number of storeys	NDBF		DBF-100		DBF-80		DBF-60	
	A	B	A	B	A	B	A	B
2	-	-	-	-	-	-	-	-
4	-	1	-	-	-	-	-	-
8	1	2	-	-	-	1	-	1
12	2	3	-	1	-	1	1	1

Figure 6a must be examined together with the information presented in Table 3 as dynamic instability in the structures resulted from large storey drifts developing in a single storey or in two consecutive storeys along the building height. In Fig. 6b, the coefficient of variation (COV) of the peak storey drifts along the height of the structure are also given. This additional parameter allows one to assess the distribution of the lateral deformation over the structure height and, therefore, the tendency of the frame to form a soft storey mechanism.

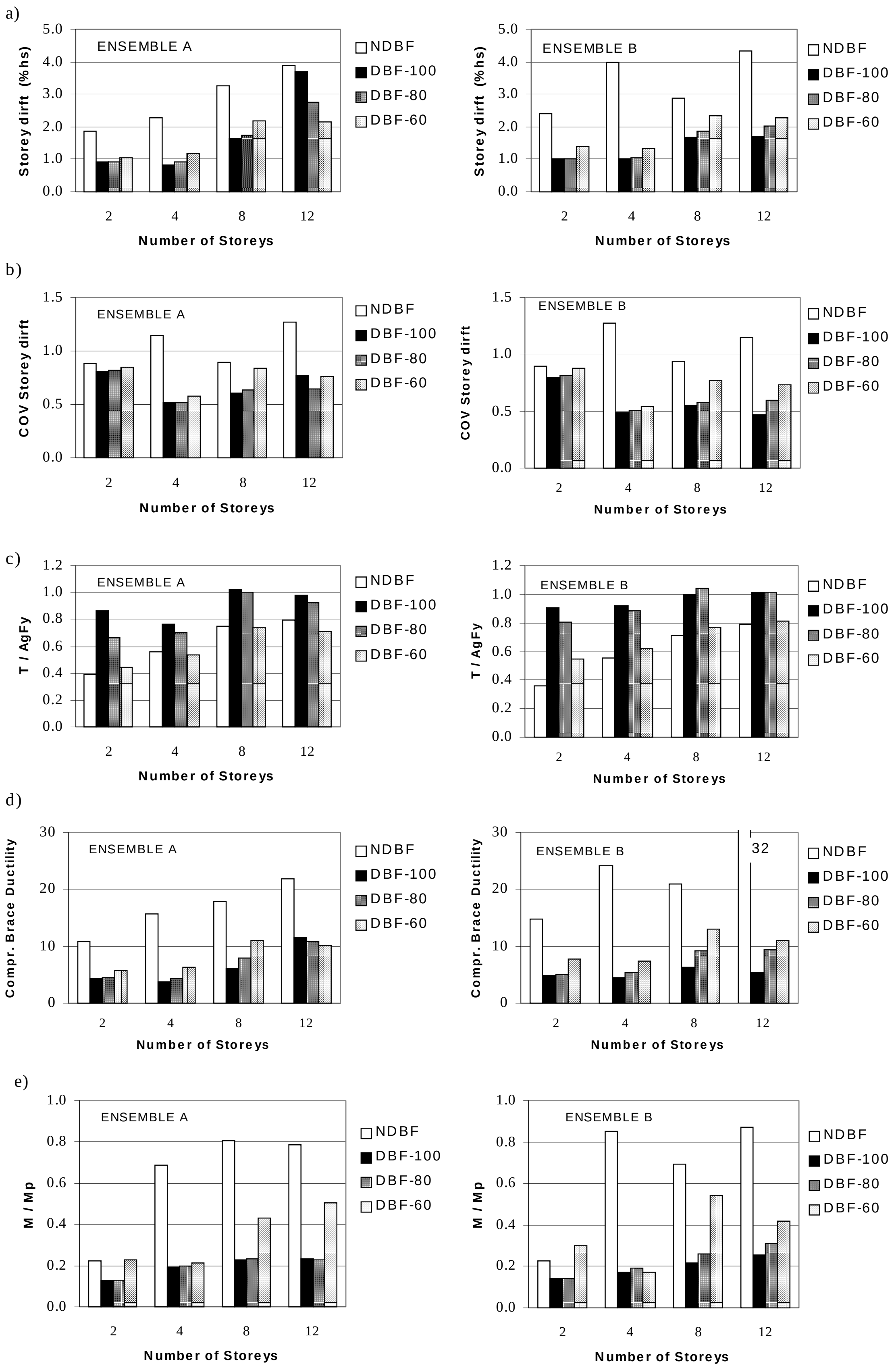

Figure 6: Results of the analyses (M+SD) for ground motion ensembles A and B: a) Storey drift; b) COV of storey drift; c) Tension brace load; d) Compression brace ductility; e) Bending moment in columns

Figure 6a shows that the maximum peak storey drift generally increases with the building height. NDBF structures also generally undergo much larger deformations than DBF structures, with the 4-storey and taller NDBFs experiencing storey drifts beyond the 0.02 hs NBCC limits. Table 3 shows that the NDBF systems could not withstand all ground motions of ensemble A for the 8- and 12-storey buildings and that the 4-storey NDBFs collapsed under one ground motion of ensemble B. Figure 6b also indicates that lateral deformations in NDBFs are more concentrated along the building height than in DBFs. These observations suggest that the NDBF system would not be appropriate for buildings of four storeys or taller.

Storey drifts in 8-storey and lower DBF structures remained within the NBCC limits under ensemble A records. For these structures, the deformations increase slightly when the design brace tension load is reduced, as a result of the greater flexibility and lower flexural strength of the beams used in the DBF-80 and DBF-60 frames. Under the same ground motions, the 12-storey DBF buildings experienced storey drifts in excess of 0.02 hs and one occurrence of instability was observed for the DBF-60 frame. Under the ensemble B earthquakes, the 2- and 4-storey DBF systems behaved well while dynamic instability developed in the 8-storey DBF-80 and DBF-60 as well as in all three 12-storey DBF buildings. As mentioned earlier, the lateral deformation demand in DBFs was generally more evenly distributed over the frame height than in NDBFs. Higher demand concentration is noticed, however, in the 2-storey DBF frames. This is due to the fact that inelastic action in these structures only developed in the first storey because the top floor braces were significantly over-designed to meet the b/t and KL/r limits. Such concentration is not detrimental as these structures remained stable and the bending moments in the columns, which are discussed later, remained small. Thus, based on storey drift considerations, DBFs generally performed better than NDBFs and should be permitted for structures up to 8-storey in height. For the 2- and 4-storey structures, the beams can be designed for an unbalanced vertical brace forces determined with a tension brace load equal to 60% of the full tensile yield resistance of the braces. For the 8-storey buildings, a DBF-100 design is preferred as this would lead to a more robust response under severe ground motions.

As expected, the computed maximum brace tension forces (Fig. 6c) are lower in the NDBF system due to the limited capacity of the beams to resist the brace forces that develop after buckling of the braces. For the same reason, tension brace forces are also generally higher in the DBF-100 buildings than in the DBF-80 and DBF-60 structures. This trend is also consistent with the storey drift results described earlier. The increased performance of chevron braced frames with stronger beams is mainly due to their ability to develop higher brace tension loads which minimises the degradation of the storey shear resistance that follows the buckling of the braces. On the other hand, these higher brace tension loads must be accounted for in the design of the brace connections and column splices, which can have a significant impact on the cost of the structure. For the DBF-100 system, the maximum expected brace tension force can be taken equal to the yield load of the braces. For the other DBF systems, that force can be lower, as shown in Fig. 6c, and a conservative estimate can be obtained from a study of the yielding mechanism of the frame, as described by Robert [1999].

Rectangular HSS bracing members are prone to low-cycle fatigue failure under cyclic inelastic loading. The fracture life depends on the applied loading history and the b/t and KL/r properties of the braces. Tests by Archambault et al. [1995] showed that compact HSS braces with a slenderness ratio varying between 60 and 100 fracture at a ductility of 3 to 4 when subjected to stepwise increasing symmetrical deformation cycles. In chevron braced frames, however, inelastic deformations in braces mainly develop in compression, which is less critical. For instance, Lee and Goel [1987] and Liu [1987] showed that well detailed braces can survive several cycles at a compression ductility up to 15-20 prior to fracture if subjected to limited tension yielding. Figure 6d shows that brace compression ductility generally increases with the building height. NDBFs exhibit higher ductility demand but their braces did not yield in tension (Fig. 6c). For these structures, excessive brace ductility developed, however, in the 12–storey frames under ensemble A. Brace tension yielding for the DBFs only developed in the 8- and 12-storey DBF-100 and DBF-80 buildings. The brace compression ductility levels that were reached in all three 2- and 4-storey DBF structures as well as in the 8- and 12-storey DBF-60 frames are then well below the brace inelastic capacity. The brace compression ductility demand in the 12-storey DBF-100 and DBF-80 structures appears to be excessive, considering that tension yielding developed in the braces. For the 8-storey DBF-100 and DBF-80, the computed compression ductility is near critical as the tension ductility levels in these structures reached respectively 3.7 and 1.9. The proposed building height limitations, based on storey drift, can be used to avoid the cases where brace fracture is likely for both the NDBF and DBF systems.

Maximum bending moments observed in the columns of the bracing bents are given in Fig. 6e. This flexural response is the result of the differential storey drift that develops in each pair of consecutive storeys corresponding to each of the column tiers. The study indicates that very high bending moments are induced in the 4-storey and taller NDBF structures. Although limited in amplitude, several occurrences of plastic rotation were observed in the columns of these frames. Such high flexural demand can lead to failure of the columns if not properly accounted for in design. The computed bending moments for all 2-storey buildings as well as for the DBF-100 and DBF-80 structures are approximately equal to 20% of the column flexural capacity, which should not be detrimental to column response. Higher moments were observed, however, in the 8- and 12-storey DBF-60 buildings. Again, the cases where undesirable large bending moments were observed can be avoided if the building height limits proposed for lateral deformation are used.

CONCLUSIONS

Nonlinear dynamic analyses have been performed on 2-, 4-, 8-, and 12-storey chevron (inverted V) braced steel frames subjected to earthquake ground motions. Four different design procedures were used to evaluate the influence of varying the strength of the beams on the seismic response. The NDBF frames were designed according to current CSA-S16.1 provisions, with no special requirements to control the inelastic response in the beams after buckling of the braces has occurred. The DBF structures were designed for reduced lateral seismic loads but their beams were sized to carry the forces due to the gravity loads acting together with unbalanced brace forces that develop after brace buckling. For these unbalanced brace force condition, 100%, 80%, and 60% of the brace tension yield resistance (AgFy) were considered in the DBF-100, DBF-80, and DBF-60 designs, respectively. The results indicate that a stable inelastic response with acceptable level of storey drifts, inelastic brace deformation, and secondary column bending moments can be achieved in the 2-storey NDBF structures and in DBF frames up to 8 storeys in height. For the 2- and 4- storey DBF structures, a tension brace load of 0.60 AgFy can be used in the design of the beams while 100% AgFy should be considered for the 8-storey buildings.

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