

SEISMIC DESIGN OF DUAL CONCENTRICALLY BRACED STEEL FRAMES FOR STABLE SEISMIC PERFORMANCE FOR MULTI-STORY BUILDINGS

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ABSTRACT

The paper presents an innovative dual buckling restrained braced (BRB) frame system which includes a conventional BRB frame coupled with a common braced steel frame. The latter is designed to remain essentially elastic to redistribute the inelastic demand in the buckling restrained brace elements over the entire building height, thus mitigating the likelihood of soft-story response. Design guidelines for this system are presented in the paper. The performance of the system is then validated through incremental dynamic analysis for structures between 8 and 24 stories in height. Conventional BRB frames are also studied for comparison purposes. All structures are designed according to the latest Canadian seismic provisions. Both intra-plate and inter-plate (subduction) ground motion time histories are considered in the analytical study. The results show that the proposed dual BRB frame system provides a better performance against structural collapse than conventional BRB frames. Subduction earthquakes are also found to be more critical than intra-plate events.

Introduction

Buckling restrained braces (BRBs) have recently been introduced in North America as a substitute for conventional steel braces in concentrically braced steel frames. These braces are typically made of a steel plate inserted in a steel tube filled with concrete and an unbonded material is used between the steel core and the concrete fill (Fig. 1a). The steel plate is thus prevented from buckling and can yield both in compression and tension, which leads to a much more stable and symmetrical inelastic response under cyclic demand without strength degradation due to buckling (Watanabe et al. 1988; Tremblay et al. 1999; Iwata et al. 2000; Clark et al. 2000). Several analytical studies have shown that BRB frames as illustrated in Fig. 1b can exhibit satisfactory performance in low-rise applications (Ko et al. 2002; Sabelli 2003; Tremblay et al. 1999, 2004). High ductility demand levels as well as concentrations of storey drift demand and dynamic instability have however been reported in this type of frames, especially in taller buildings (Fahnestock et al. 2003; Mayes et al. 2004; Tremblay and Poncet 2004).

A dual buckling restrained braced (Dual BRB) frame system has been proposed by

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Tremblay (2003) to mitigate this effect and ensure stable inelastic response for multi-storey structures. Compared to the conventional BRB framing of Fig. 1b, BRB members in a dual system are only used in one half of the bracing bent width, as shown in Fig. 1c, and the braces in the second half of the bay width are designed to remain essentially elastic under a strong ground shaking. The edge columns and beam segments that connect to those stronger braces are also designed to avoid yielding and a continuous vertical elastic truss is formed by adding a central column to the bracing bent. The role of this vertical truss is to force an inelastic response of the structure that approaches the first vibration mode, thus preventing the concentration of inelastic response over the building height and mobilizing the entire energy dissipation potential of the structure. In the Dual BRB frame system, additional steel tonnage is required to construct the elastic, stronger vertical truss but only half the number of BRB members is needed.

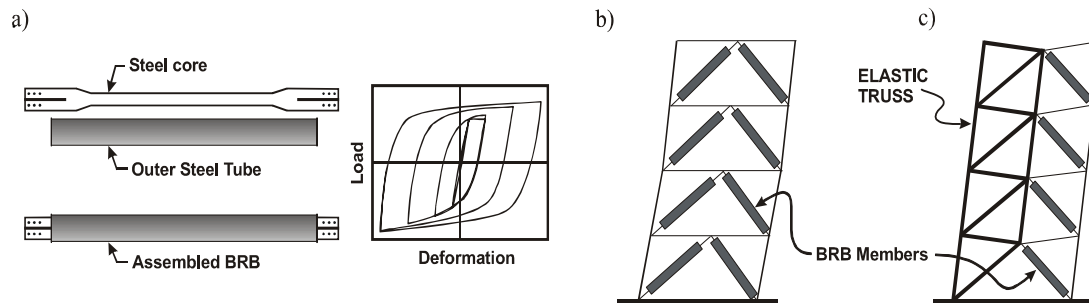


Figure 1. Structural systems and brace hysteretic response: a) typical buckling restrained brace (BRB) member and hysteretic response; b) BRB frame; c) Dual BRB frame.

Tremblay and Poncet (2004) confirmed the good performance of the Dual BRB system for building structures up to 16 stories when subjected to crustal and sub-crustal earthquakes. Tremblay and Merzouq (2004) further studied the system using more realistic design and modeling assumptions for their structures. The buildings had between 8 and 24 stories in height and were located in Victoria, B.C., a site located in the vicinity of the Cascadia subduction zone. For each building, both the conventional and the Dual BRB frame systems were studied and all structures were designed using the modal response spectrum analysis method. The BRB inelastic response was reproduced using a Ramberg-Osgood formulation including kinematic and isotropic strain hardening properties. The structures were subjected to an ensemble of site representative ground motions from crustal and sub-crustal earthquakes as well as simulated and recorded ground motion time histories from **M**8.5 subduction events. This paper presents a subsequent phase of this study in which a new geometry and a revised design procedure were used for the Dual BRB frame buildings. The sample structures were subjected to the same intra-plate and inter-plate (subduction) ground motion sets to evaluate their seismic performance. Comparison with the response of the conventional BRB frames is also given in the paper.

Buildings studied

Building description and properties

Five prototype office buildings assumed to be located in Victoria, B.C., are studied. The structures have 8, 12, 16, 20, and 24 floors in height, with a uniform storey height of 4.0 m. They

have a rectangular foot print with plan dimensions of 45.5 m x 72.5 m, as shown in Fig. 2a. Earthquake effects acting parallel to the long walls were examined. In that direction, one two-bay bracing bent is used along each of the two exterior walls, which results in symmetrical structures with no in-plane eccentricity. Figure 2b and 2c show the chevron bracing configuration adopted by Tremblay and Merzouq(2004) for the conventional and Dual BRB frame systems, respectively. One elastic truss was used in each of the two bracing bents. As shown in Fig. 2d, a split-X geometry was selected for the Dual BRB frame in this study, with a single elastic truss located in the central portion of the two-bay bracing system. With this new configuration, the elastic truss exhibits a higher flexural stiffness for a given amount of steel. Because of its central pinned connection, the elastic truss does not attract additional overturning moment forces compared to the conventional BRB frame, but can still limit inelastic demand concentration due to higher mode response. In this modified configuration, earthquake induced axial loads in the edge columns are bounded by the capacity of the BRB members, which is more effective than having the edge columns being part of the elastic trusses, as was the case in Fig. 2c.

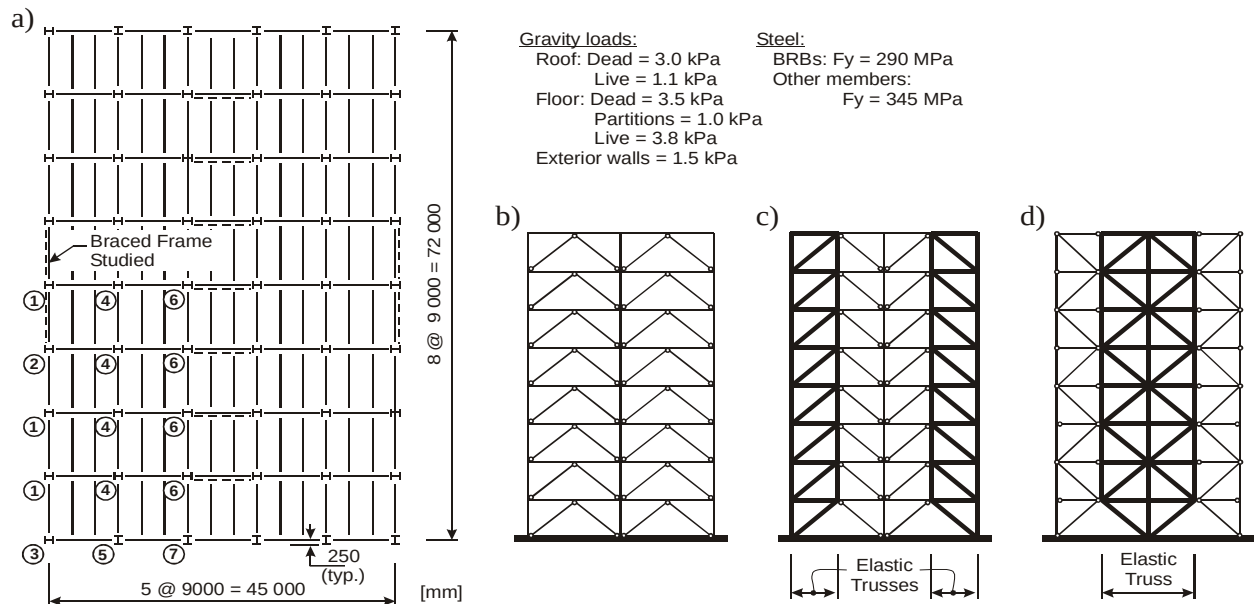


Figure 2. a) Plan view of the buildings studied; b) 8-story BRB frame; c) 8-story Dual BRB frame (Tremblay and Merzouq 2004); d) 8-story Dual BRB frame (this study).

The structures were designed according to the provisions of the 2005 National Building Code of Canada (NBCC). Actions from earthquakes were obtained using the response spectrum analysis method and the forces were reduced by a ductility- and overstrength-related force modification factors, $R_d = 4.0$ and $R_o = 1.3$. These forces were then adjusted such that the resulting base shear force from response spectrum analysis was equal to 80% of the earthquake lateral force from the code static equivalent force procedure, as specified in NBCC 2005 for regular structures. Table 1 gives the building seismic weight, W , the computed period of the buildings in their first three vibration modes, T_1 , T_2 , and T_3 , and the design base shear ratio V/W . In the static force method, it is noted that the fundamental period of the structure must not exceed $T_a = 0.05 h_n$, where h_n is the height of the structure, and that the base shear force must

not be less than the value computed with a period of 2.0 s. Because of these two limitations, the same base shear ratios had to be used for both bracing systems, as well as for all 12-storey and taller buildings, even if their computed fundamental periods were different. Thus, several structures possessed some inherent overstrength due to code prescribed requirements.

Table 1. Buildings properties.

<i>n</i> floors	<i>W</i> (MN)	BRB frames				Dual BRB frames			
		<i>T</i> ₁ (s)	<i>T</i> ₂ (s)	<i>T</i> ₃ (s)	<i>V/W</i> (%)	<i>T</i> ₁ (s)	<i>T</i> ₂ (s)	<i>T</i> ₃ (s)	<i>V/W</i> (%)
8	114	2.22	0.81	0.50	4.00	1.50	0.49	0.34	4.00
12	172	3.48	1.24	0.72	2.77	2.46	0.74	0.42	2.77
16	231	4.28	1.48	0.84	2.77	3.18	0.98	0.52	2.77
20	289	5.02	1.69	0.93	2.77	3.85	1.20	0.63	2.77
24	348	5.62	1.93	1.09	2.77	4.52	1.39	0.73	2.77

For both frame systems, the BRB members were designed in the first place to develop a factored resistance equal to the code required strength. Capacity design principles were then applied to size the remaining members of the braced frames so that they could resist the maximum forces likely to develop under strong ground shaking. For the Dual BRB system, member forces in the elastic truss system depend on the stiffness of the elastic truss as well as the variation between adjacent floors of the brace plastic deformations experienced during earthquake ground motions. Knowledge of the inelastic response of the structure is therefore essential to evaluate these forces, as can be obtained using nonlinear dynamic analysis of the structures. From the analyses performed for the Dual BRB frames of Fig. 2c, Tremblay and Merzouq (2005) proposed empirical guidelines for the design of the elastic truss members. These rules were used for the design of the modified Dual BRB frames of Fig. 2d, as briefly exposed herein. For the elastic braces, the design axial load, P_{EB} , was taken as varying linearly from 0.95 $P_{EB,RS}$ at the building top to 0.70 $P_{EB,RS}$ at the building base, where $P_{EB,RS}$ is the elastic brace force obtained from an elastic response spectrum analysis. For the exterior columns of the elastic truss, the design compression force at a level i , $C_{CC,i}$, was obtained from:

$$C_{CC,i} = \sum_{j=i}^n \alpha_j \gamma_j (P_{EB,j} - P_{BRB,j}) \sin \theta_j, \quad \alpha_j = 1 + 0.5[(j-1)/(n-1)]^\beta \quad (1)$$

In this equation, P_{BRB} is the yield capacity of the BRB members, including strain hardening and frictional effects, taken equal to 1.35 times the brace nominal yield load. The correction factor, α , is applied to account for the fact that P_{EB} and P_{BRB} at a given floor do not necessarily reach their maximum at the same time. In the equation for α , the exponent β varies linearly from 1.0 for the 24-storey building to 3.0 for the 8-storey building. Time lag between maximum P_{BRB} and P_{EB} values is a consequence of higher mode response in the structures and, therefore, the factor α increases when j approaches the number of floors, n , or when n is increased. In Equation (1), the multiplier γ is used to capture the variation in the brace forces in adjacent stories. Its value varies between 0.0 and 1.0 and the resulting force distribution is shown in Fig. 3. The calculation of the forces C_{CC} must be performed twice, from the building top and from the building base, and the lower value is kept for design, as illustrated in Figs. 3 and 4.

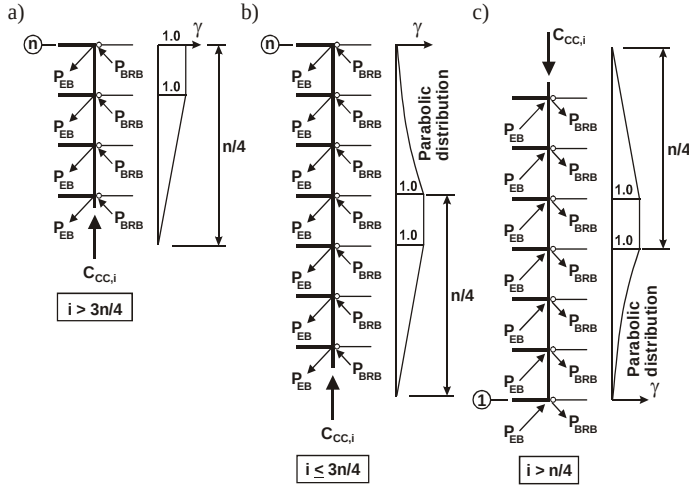


Figure 3. Brace force patterns for determining central column forces C_{cc} in the: a) and b) Upper levels; c) Lower levels.

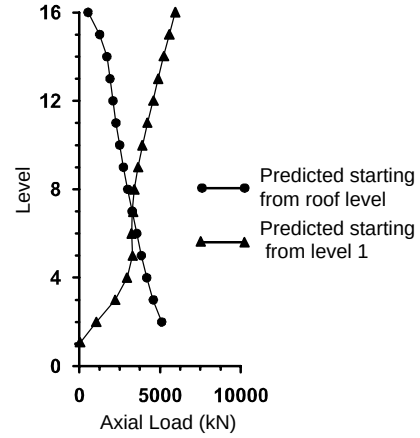


Figure 4. Predicted C_{cc} values for the 16-story building.

The beam segments that are part of the elastic truss were designed for an axial load equal to the horizontal components of the design axial load, P_{EB} . For the edge columns, the design axial load was calculated by simply summing up the vertical component of the P_{BRB} forces acting above the level under consideration, thus representing the observation in previous analyses that all BRB members could yield at the same time over the entire building height.

Analysis procedure

Modeling Assumptions

Nonlinear time step dynamic analysis of the structures was performed using the Ruaumoko computer program (Carr 2002). Torsional response and in-plane diaphragm deformations were neglected, and a two-dimensional numerical model could therefore be used to determine the seismic response of the symmetrical building structures. The model included one two-bay bracing bent acting in the direction parallel to the long walls together with all leaning tributary gravity columns. The BRB members were modeled using a symmetrical Ramberg-Osgood formulation with isotropic and kinematic strain hardening properties adjusted based on test results. A brace yield capacity in compression 10% larger than the tension yield strength was specified to account for frictional response. The BRB members were considered as pin-connected to the other framing members. Beams and columns, as well as the braces in the elastic trusses of the Dual BRB system, were modeled using frame elements with bilinear plastic hinges concentrated at their ends. These elements were also capable of axial yielding in tension and compression. The yield strength in compression was set equal to the compression resistance of the members, including buckling effects. Perfectly rigid beam-to-column connections were assumed at all joints. The columns of the bracing bents were assumed to be fixed at their bases whereas pinned connections were considered at the base of the gravity columns. The column splice connections were assumed to be fully rigid in flexure. At every level, the lateral displacements of the gravity columns were constrained to be the same as the lateral deformations of the bracing bents. A Newmark constant acceleration integration scheme with a maximum time

step of 0.001 s was used. Large displacement analysis was performed with 100% of the dead load, 50% of the floor live load, and 25 % of the roof snow load applied on the structure. Rayleigh damping proportional to initial stiffness was used with 3% of critical damping in the first and third vibration modes.

Two earthquake record ensembles were used in the study. The first set 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 Victoria region: M6.5 at 30 km and M7.2 at 70 km (Tremblay and Atkinson 2001). In the second ensemble, four ground motion time histories were simulated for four possible M_w 8.5 rupture scenarios along the Cascadia subduction fault plane. The ground motion amplitude was adjusted to match, on average, the 2% in 50 year probability of exceedance spectrum over the applicable period range.

Analysis results

Overall Behavior and response under Design Level Earthquakes

Table 2 summarizes the occurrences of collapse by dynamic instability for each building height, frame system and ground motion ensemble. In this paper, collapse by dynamic instability is said to have occurred when the story drift in any given floor exceeded 5% of the story height. Collapse of the structures was due to gravity loading acting on the deformed structures. Failure of the conventional BRB frames occurred under some of the intra-plate earthquakes and all inter-plate earthquakes. Short-period structures were found to be sensitive to collapse, which is attributed to the lower demand-to-supply ratio for the taller buildings resulting from code design requirements. Collapse of the Dual BRB system only occurred once under a Cascadia motion. However, even in that case, the elements of the elastic truss remained elastic, as it was also the case under all other ground motions.

Table 2. Seismic performance of the BRB and Dual BRB bracing systems.

n floors	Occurrences of dynamic instability				Confidence level (%) against collapse ¹			
	BRB frames		Dual BRB frames		BRB frames		Dual BRB frames	
	Crust. (/10)	Subd. (/4)	Crust. (/10)	Subd. (/4)	Crust.	Subd.	Crust.	Subd.
8	3	4	0	0	< 1.0	-	99.9	80.0
12	5	4	0	1	< 1.0	-	98.4	89.0
16	3	4	0	0	< 1.0	-	99.9	41.3
20	0	4	0	0	63.7	-	99.9	86.7
24	0	4	0	0	70.5	-	99.3	89.8

¹Parameters used in calculation: $\beta = 0.25$ for ϕ_u , $\gamma_a = 1.1$, $k = 2.39$, $b = 1.0$ and $\beta_{UT} = 0.5$

Figure 5 shows the response of the 12- and 24-story buildings under one intra-plate and one inter-plate earthquake. As shown in Figs. 5a and 5b, the 12-story conventional BRB frame failed under both the intra-plate and the subduction Cascadia motions. The 24-story BRB frame remained stable under the intra-plate earthquake but experienced a similar collapse response as the 12-story building under the long duration Cascadia earthquake B. Only the 12-storey Dual BRB frame collapsed under the intra-plate earthquake. In all cases, collapse occurred

progressively, as a result of the accumulation of permanent lateral deformations towards one direction. Figure 5b shows snap shots of the lateral building displacements near collapse (or at peak lateral roof deformation when collapse was not observed). The ‘S’ shape deformation observed for the conventional BRB frames confirm that higher mode response could easily develop in the inelastic range, leading to significant concentration of inelastic demand over the building height. Conversely, the Dual BRB frames exhibited a deformation pattern that remained nearly perfectly straight from the foundation to the roof level, mimicking first mode response, even when structural collapse occurred. This indicates that the elastic truss behaved as intended, acting as a back bone preventing any marked deviations in the inelastic story drift demand. From Table 2, it is clear that inter-plate earthquakes are more critical for dynamic instability. This is mainly attributed to their longer duration of strong shaking, which represents a more favorable condition for the progressive build-up of one-sided plastic deformations.

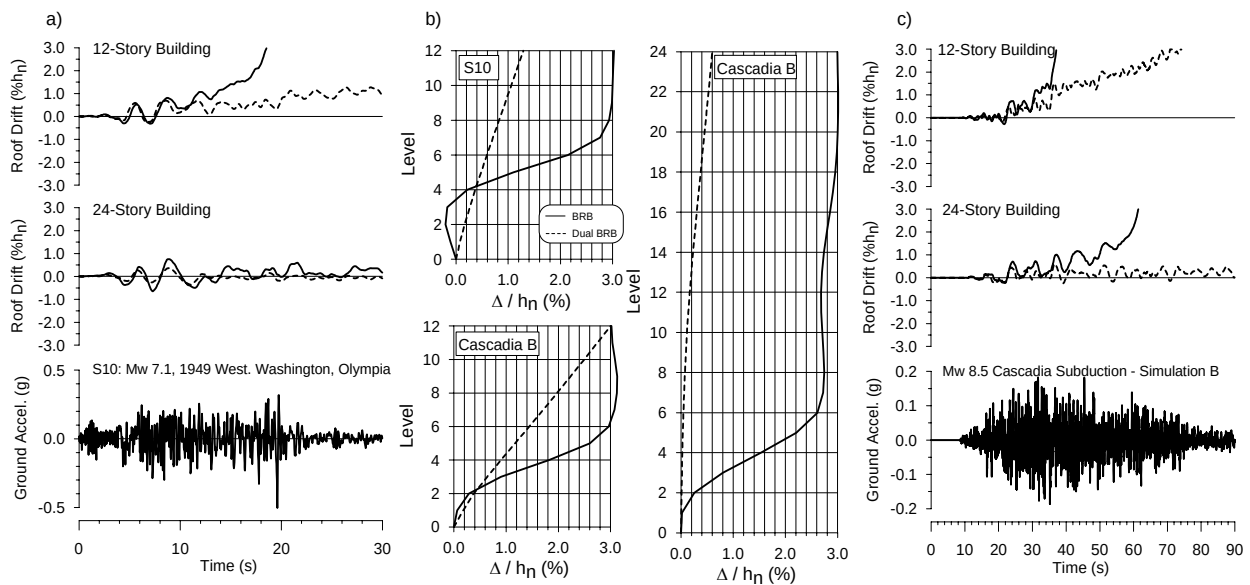


Figure 5. Response of the 12- and 24-story buildings under crustal ground motion S10 and **M8.5** Cascadia subduction ground motion B: a) Time history of lateral roof deformations under motion S10; b) Laterally deformed shape near collapse (or at maximum roof deformation when no collapse occurred); c) Time history of lateral roof deformations under Cascadia motion B.

Peak story drifts and peak horizontal floor accelerations are used to assess and compare the response of the two framing systems. The story drift is expressed as a percentage of the story height, h_s , whereas the floor accelerations are given as a fraction of the peak ground acceleration (PGA) of the applied ground motions. Story drift values can be directly compared to NBCC limits of 2.5% h_s . They represent a good measure of the damage to the framing system, building façades and windows, partitions, elevators, and mechanical equipment. Floor accelerations create damage to ceiling and light systems, the electrical and mechanical equipment, and the building contents. The variation of the peak story drift along the building height was also used in this study to evaluate the extent of concentration of the inelastic demand. For each ground motion, the coefficient of variation (COV) of the peak story drift values over the height of the structure

was computed to assess this behavior. Finally, the peak strains in the core of the BRB elements, ϵ_c , were recorded as a measure of the plastic damage experienced by the brace core material. Calculations of the peak strain values accounted for the actual BRB core length as well as brace end and connection size effects, as described in Tremblay and Merzouq (2004).

The 50th percentile values of these maximum response parameters were determined for each ground motion ensemble and are presented in Fig. 6. For the peak story drifts, floor accelerations and brace strains, the statistics were computed using the maximum of the peak values over the building height for each ground motion. Results for the collapsed buildings were omitted in the calculations and cases for which values are missing are indicated by an asterisk (*). As all BRB frames collapsed under all four subduction earthquakes, statistical results are not presented for these cases. In Fig. 6a, the use of a Dual BRB system resulted in generally lower and more consistent peak story drift values under the intra-plate earthquakes. All values under both types of ground motions met the code limit. As anticipated from previous discussions, the BRB frames experienced more pronounced variations in story deformations along their height. Figure 6b indicates that the likelihood of damage concentration increases with the building height, even if the Dual BRB system is adopted. Peak acceleration ratios are generally higher for the Dual BRB frames, which can be attributed to their shorter periods of vibration (Table 1) and to the presence of the elastic truss that likely created larger horizontal forces. Subduction earthquakes induced relatively larger floor accelerations in the structures. However, the acceleration levels computed in all cases do not vary much with the building height and are remarkably close to the PGA values. The brace core deformation demand in Fig. 6c is consistent with the peak story drift results discussed earlier. Under the intra-plate earthquakes, the ϵ_c values are lower for the Dual BRB system despite the fact that this system only includes half the number of BRB elements, indicating a more efficient use of the available energy dissipation capacity.

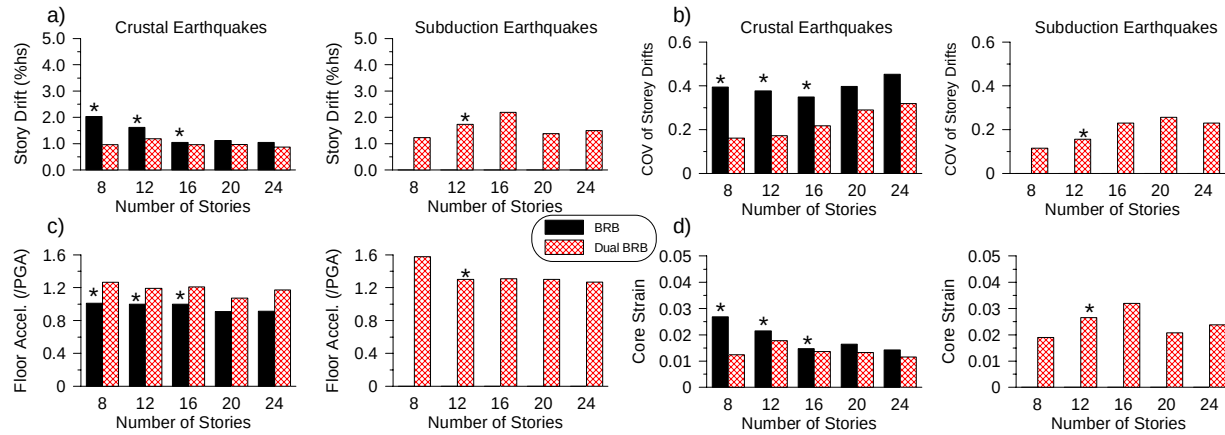


Figure 6. Statistics on the performance of the structures (50th fractile): a) Maximum peak story drifts; b) COV of peak story drifts; c) Maximum peak horizontal floor accelerations; d) Maximum brace core strain demand.

Incremental Dynamic Analysis Results

Nonlinear incremental dynamic analysis (IDA) was also performed for both BRB frames

(Vamvatsikos and Cornell 2002). The ground motion amplitude was multiplied by a scaling factor, S_F , that was increased in successive 0.1 increments from $S_F = 0.2$ up to the collapse of the structures. It is noted that the case $S_F = 1.0$ corresponds to the design ground motion level. The 50th percentile IDA curves obtained for each ground motion ensemble are plotted in Fig. 7. These plots curves demonstrate the benefits of using a Dual BRB system and the relatively higher demand imposed by the long duration subduction earthquakes. Clearly, the BRB frames do not possess enough capacity to sustain either type of ground motions. For the Dual BRB system, the reduction in capacity is significant when subduction earthquakes are applied in lieu of the intra-plate earthquakes. Using the IDA curves, the confidence level that the different structures will not collapse under a given ground motion ensemble can be estimated following the procedure that was developed for steel moment resisting frames in the FEMA 350 document (FEMA 2000). The assumptions adopted by Tremblay and Merzouq (2004) were used again in this study. In particular, the global drift capacity of the structures was defined as the point on the IDA curve where the slope became less than 20% of the initial elastic slope. The computed results are presented in Table 2. The calculations were not performed for the BRB frames under the Cascadia subduction earthquakes, as these structures failed under the design ground motion level, clearly indicating unsatisfactory performance. In the FEMA procedure, adequate performance is achieved if the level of confidence is equal to or exceeds 90 %. None of the BRB frames reached this minimum value under the intra-plate earthquakes. The use of a Dual BRB system for this seismic hazard was found to provide satisfactory performance for all building heights. However, the Dual BRB system did not exhibit sufficient robustness under the subduction earthquakes.

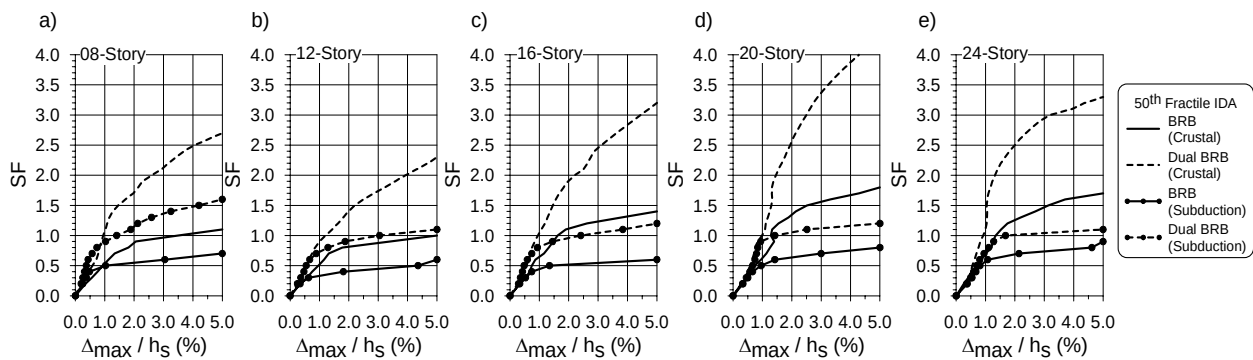


Figure 7. Ground motion amplitude-maximum peak story drift (50th fractile) relationship for the: a) 8-story building; b) 12-story building; c) 16-story building; d) 20-story building; e) 24-story building.

Conclusions

The analytical study presented in this paper showed that multi-story concentrically braced steel frames located in Victoria, B.C. did not exhibit satisfactory seismic performance for the Victoria, even if built with buckling restrained braces with stable hysteretic response. The structures experienced large story drifts and several occurrences of dynamic instability were observed, indicating that the frames did not possess sufficient capacity to redistribute the inelastic demand over their height. Dual buckling restrained braced frames with back-up elastic truss response were found to behave satisfactory under ground motions induced by intra-plate

crustal and sub-crustal earthquakes. Their performance was not adequate, however, under long duration subduction ground motions, as those anticipated along the Pacific west coast of Canada.

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