

ASSESSMENT OF SEISMIC DESIGN FORCES IN DUAL BUCKLING RESTRAINED BRACED STEEL FRAMES

VALUTAZIONE DELLE FORZE SISMICHE DI PROGETTO IN TELAI IN ACCIO ACCOPPIATI CON CONTROVENTI AD INSTABILITA' CONTROLLATA.

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ABSTRACT

A study was conducted on a dual buckling restrained braced frame system for multi-storey buildings. In this system, half of the bracing bent forms an elastic vertical steel truss which is added to mitigate the concentration of inelastic demand over the building height and achieve stable inelastic seismic response. The response of typical structures is examined under crustal and sub-crustal seismic events as well as long duration subduction earthquakes. The building height varies from 8 to 24 storeys and the structures are designed according to Canadian seismic provisions. The performance of the system is presented and discussed. The analysis results are then used to develop guidelines for the design of the elastic truss components.

SOMMARIO

Uno studio è stato condotto su un sistema strutturale a telaio con controventi ad instabilità controllata per la realizzazione di edifici multi-piano. In questo sistema, metà della struttura portante controventata forma un puntone verticale in acciaio che è preposto alla moderazione della concentrazione di deformazione plastica richiesta lungo l'altezza dell'edificio ed ottenere una stabile risposta sismica in campo plastico. La risposta di strutture tipiche è esaminata tanto sotto l'azione di eventi sismici "crostali" e "sotto-crostaali" quanto sotto l'azione di terremoti di subduzione. L'altezza degli edifici varia da 8 a 24 piani e le strutture sono progettate in accordo alle prescrizioni sismiche della norma Canadese. La prestazione del sistema è presentata e discussa. I risultati delle analisi sono utilizzati per sviluppare indicazioni per la progettazione dei componenti elastici che formano il sistema a puntone.

1 INTRODUCTION

Buckling restrained braces (BRBs) have recently been used in concentrically braced steel frames for enhanced seismic performance. These braces are made of steel plates inserted in steel tubes filled with concrete. An unbonded material is used between the steel core and the concrete fill such that the axial load is entirely resisted by the steel plate which can yield in tension and in compression, without buckling. This results in a stable and symmetrical inelastic response under cyclic inelastic demand without strength degradation [1]. However, recent studies have shown that despite the stable hysteretic brace response, multi-storey BRB

frames can exhibit storey response with concentration of inelastic deformation along their height [2].

The dual buckling restrained brace frame system (Dual BRB) shown in Fig. 1c was proposed [3] to mitigate this effect and ensure stable seismic inelastic response. In this system, BRB members are used only in half of the bracing bent width, the braces in the second half of the bay width being designed to remain essentially elastic under strong ground shaking. The edge columns and beam segments that connect to those stronger braces are also designed to avoid yielding and a central column is added to form a continuous elastic vertical truss to force inelastic response approaching the first vibration mode, thus preventing concentrated inelastic response over the building height and mobilizing the entire energy dissipation potential of the structure. The seismic response of BRB and Dual BRB frames was compared through a parametric study involving incremental dynamic analysis [4]. The results of this study are summarised in the first part of the paper. In the second part, the force demand on the elastic truss components is examined to propose guidelines for the design of Dual BRB members.

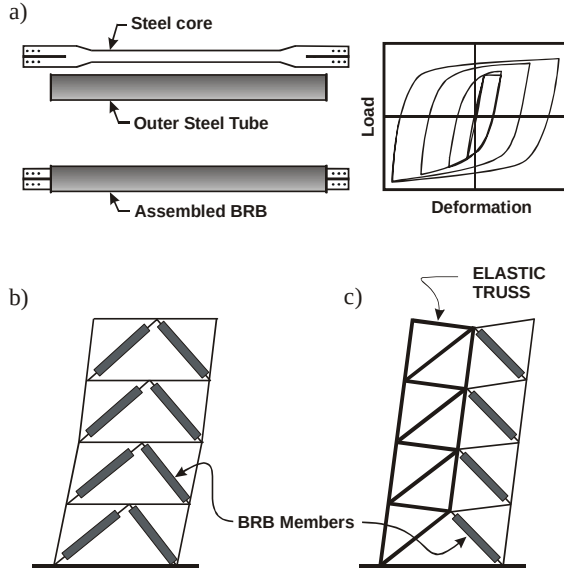


Fig. 1: BRB frames; a) BRB member; b) BRB frame; and c) Dual BRB frame.

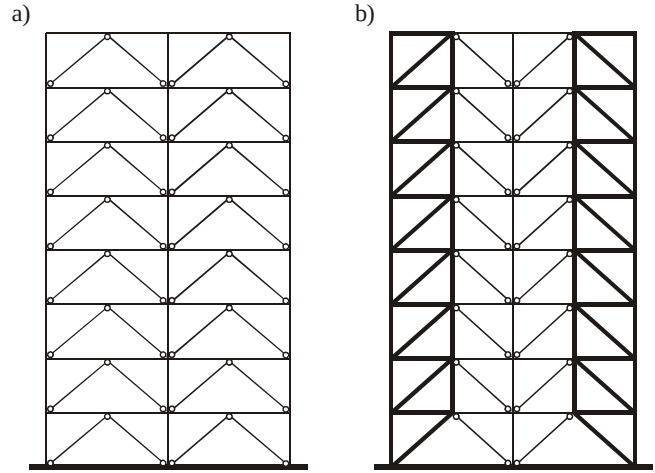


Fig. 2: Eight-story braced frame elevations: a) conventional BRB frames; Dual BRB frames.

2 PERFORMANCE OF DUAL BRB FRAMES

2.1 Building design and analysis

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. 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. The geometry of the conventional and Dual BRB frames are shown for the 8-story building in Fig. 2. The structures were designed according to the 2005 National Building Code of Canada (NBCC) [5]. 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 $R_o = 1.3$. The buckling restrained brace elements in both systems were first designed to develop a factored resistance equal to the code required strength. Capacity design principles were then applied to size the beams and the columns of the braced frames, as well as the elastic truss components for the Dual BRB

frames, so that they could resist the maximum forces likely to develop under strong ground shaking. Table 1 gives the building seismic weight, W , the design base shear ratio V/W , the periods in the first three vibration modes, T_1 , T_2 , and T_3 , and the maximum lateral capacity obtained from incremental static (push-over) analysis, V_{max} .

Table 1: Building properties

n floors	W (MN)	V/W (%)	BRB frames				Dual BRB frames			
			T_1 (s)	T_2 (s)	T_3 (s)	V_{max}/V ()	T_1 (s)	T_2 (s)	T_3 (s)	V_{max}/V ()
8	114	4.00	2.22	0.81	0.50	2.12	1.68	0.56	0.30	2.40
12	172	2.77	3.48	1.24	0.72	1.80	2.70	0.90	0.47	2.15
16	231	2.77	4.28	1.48	0.84	1.85	3.43	1.12	0.58	2.11
20	289	2.77	5.02	1.69	0.93	1.79	4.21	1.35	0.69	2.02
24	348	2.77	5.62	1.93	1.09	1.75	4.73	1.53	0.79	1.99

For regular structures, NBCC 2005 requires that the results from response spectrum analysis be scaled such that the base shear force is equal to or greater than 80% of the earthquake lateral force from the code static equivalent force procedure. In the static force method, the fundamental period of the structure must be limited to $T_a = 0.05 h_n$, where h_n is the height of the structure. In addition, the code design spectrum is flat for periods longer than 2.0 s. These rules resulted in the same base shear ratios for both bracing systems, even if their periods were different, as well as for all 12-storey and taller buildings. Thus, several structures possessed some inherent additional overstrength due to these NBCC code requirements.

Two earthquake record ensembles were used in the study. The first ensemble 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 [6]. 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 design spectrum over the applicable period range.

Incremental nonlinear dynamic analysis of the structures was performed using the RUAUMOKO computer program. Taking advantage of the symmetry of the buildings, two-dimensional analytical models including one two-bay bracing bent as well as the building columns stabilised by the bracing bent studied were used. The buckling restrained braces were modelled using a symmetrical Ramberg-Osgood formulation that was modified to prevent offsets of the forces in small amplitude cycles (Pyke model) and to include isotropic/kinematic (I-K) strain hardening properties. Other members were modelled using elastic beam-column elements such that the force demand could be assessed.

2.2 Seismic performance

Under the design level earthquakes, large lateral deformations developed in several buildings and structural collapse was said to have occurred when the peak storey drift angles exceeded 5%. Table 2 gives the number of collapse occurrences for each building category and earthquake ensemble. Collapse was observed in conventional BRB frames up to 16-storey under the crustal and subcrustal earthquakes. The taller frames could withstand these ground motions probably because of their higher reserve in lateral capacity resulting from code design provisions. Global instability however occurred under all long duration Cascadia subduction earthquakes. Conversely, all Dual BRB frames exhibited stable inelastic response

when subjected to the first ground motion set. Cascadia earthquakes produced structural collapse only for the 8- and 12-storey structures. Again, code limitations likely influenced the variation in response with the building height. Under the crustal and sub-crustal ground motions, the median values of the maximum peak storey drift angles at any level of the Dual BRB varied between 1.04 and 1.21%. Excluding the 8- and 12-storey structures, the corresponding values for the Cascadia earthquakes are 1.16 to 2.31%. All these drift angle results are well within the NBCC limit of 2.5%.

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

<i>n</i> floors	Occurrences of dynamic instability				Confidence level (%) against collapse			
	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	1	< 1.0	-	92.8	97.7
12	5	4	0	2	< 1.0	-	93.4	-
16	3	4	0	0	< 1.0	-	99.2	17.1
20	0	3	0	0	63.7	-	99.9	75.8
24	0	4	0	0	70.5	-	99.9	97.4

Following the FEMA-350 procedure for seismic performance assessment, the confidence level against collapse could be determined from the incremental dynamic analysis. The results are also presented in Table 2 for each braced frame system and ground motion set. A minimum level of 90% is generally considered for satisfactory performance. The values obtained confirmed the inadequacy of conventional BRB systems for multi-storey braced steel frames of 8 and more storeys. The Dual BRB system is adequate for the crustal and sub-crustal earthquake hazard. Similar conclusions can be drawn for the 8- and 24-story Dual BRB frames under the subduction earthquakes, but the 12-, 16- and 20-storey frames did not display satisfactory performance under these ground motions. Additional studies showed that increasing the design seismic loads for these structures resulted in acceptable response.

3 FORCE DEMAND ON DUAL BRB ELASTIC TRUSS COMPONENTS

3.1 Procedure used

The time history response of the Dual BRB structures was closely examined to better understand their behaviour and develop guidelines for the design of the elastic truss components. One important findings of this study was that the response of such multi-storey structures was significantly influenced by the contribution of the higher modes of vibrations. As a result, the peak force demand in the elastic truss members was found to develop simultaneously only over a portion of the building height. This is illustrated in Fig. 3 where snap shots of member force distributions along the building height are plotted at different times during a given ground motion. As can be seen, the forces at a given instant of time reach the envelope only over a small number of storeys and this observation served as a basis for the development of the design guidelines for the elastic truss system.

The 50th and 84th percentile values of the peak member forces from all 14 earthquake records were evaluated for each building. Only the results for the design level earthquakes were used, i.e. from ground motions scaled to match the 2% in 50 year design spectrum. The three collapse cases (Table 2) were excluded from the study. For the elastic truss components, gravity load effects were first removed prior to performing the statistical calculations so that the seismic induced forces could be isolated.

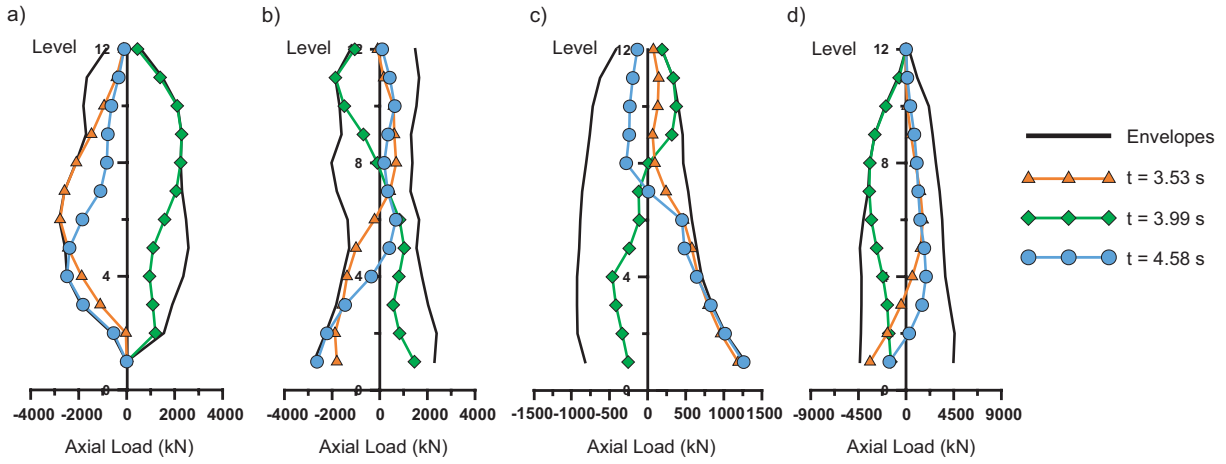


Fig. 3: Typical envelopes and snap-shots of member force demand on the 12-storey Dual BRB frame: a) Central column; b) Elastic bracing members; c) BRB members; and d) Edge columns.

3.2 Results

In design, the buckling restrained braces are first selected from code prescribed load combinations that include both gravity and earthquake forces, the latter being determined from elastic response spectrum analysis. Therefore, the BRB members can be designed following current design procedures, without information obtained from time history analysis. Forces imposed to surrounding framing members depend on the axial loads reached in the BRB elements. From the time history analysis, very consistent 50th and 84th percentile values of the BRB axial forces, $P_{BRB,50}$ and $P_{BRB,84}$, were obtained, regardless of the building height or the level along the building height, as shown in Fig. 4a for the 12- and 24-storey buildings. In the figure, A_c and F_y are the cross sectional area and yield strength of the brace core plates. For all buildings, the $P_{BRB,50}$ and $P_{BRB,84}$ values are respectively equal to 1.32 and 1.37 times $A_c F_y$. These factors reflect strain hardening response developing in the core plate material. In view of the small difference between these two numbers, a value of $1.35 A_c F_y$ was adopted for P_{BRB} for both probability levels. Table 3 gives the mean and COV value of the ratios between time history analysis results and the so predicted P_{BRB} forces.

The peak seismic axial force demand in the elastic braces, P_{EB} , is found to be closely related to the demand obtained from the elastic response spectrum method, $P_{EB,RS}$. As illustrated in Fig. 4b, that demand is however relatively higher in the top levels of the structures and a straight line variation was adopted for both probability levels. For the 50th percentile, P_{EB} varies from $0.75 P_{EB,RS}$ at the top to $0.65 P_{EB,RS}$ at the base. For the 84th percentile, the corresponding values are 0.95 and $0.70 P_{EB,RS}$. These P_{EB} values lie between the elastic force level $P_{EB,RS}$ and the ductility-reduced force level $P_{EB,RS}/R_d$ assumed in the building design ($R_d = 4.0$). This result was expected as the elastic braces at any level can resist storey shear forces beyond the storey shear required to reach yielding in the adjacent BRBs. However, the forces in the elastic braces tend to be limited when yielding develops simultaneously in BRB members of several adjacent floors.

Figure 4b shows the correlation between predicted and analysis P_{EB} results for the 12- and 24-storey buildings. Similar good agreement is found for the other structures, as indicated in Table 3. It must be noted that the P_{BRB} and P_{EB} values can be either tension or compression forces. The design of the elastic brace forces will typically be governed by compression but tension can be more critical for connections. The seismic axial load demand in the beams is simply taken as the horizontal component of the axial load predicted in the adjacent bracing member.

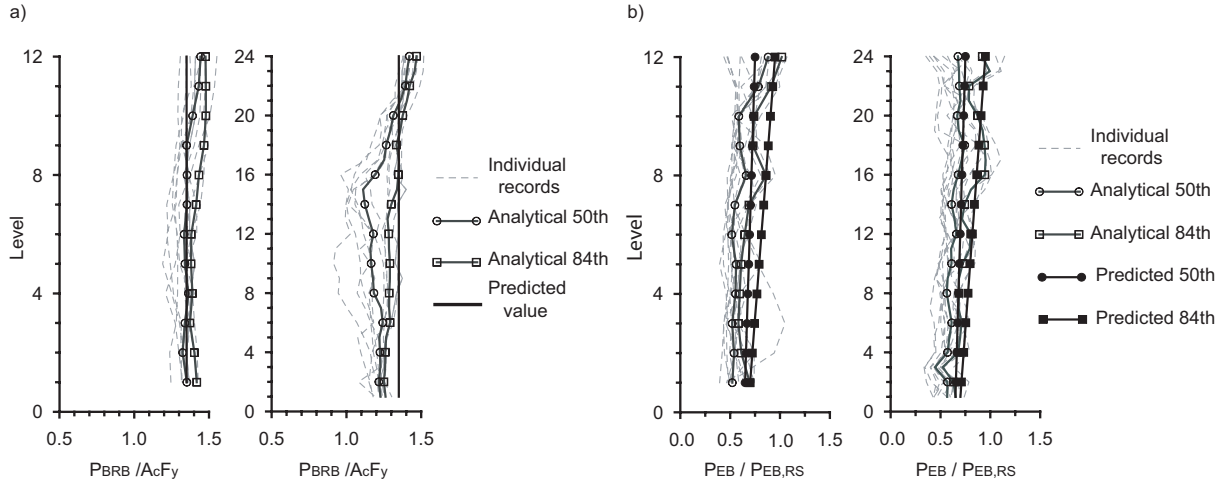


Fig. 4: Normalised brace force demand in all Dual BRB frames:
a) BRB members; b) Elastic bracing members.

Similarly to the elastic braces, central and edge column members are typically selected for compression resistance, but base anchorage or splice connections may need to be checked for tension. For simplicity, only the compression force demand is presented and discussed herein. The central columns are those located on the inner side of the elastic vertical trusses. Earthquake forces in these members result from the differences between P_{EB} and P_{BRB} . In the upper levels, maximum compression in the central column, C_{CC} , will then be equal to the cumulated differences between tension forces P_{EB} and compression forces P_{BRB} , as shown in the Figs. 5a and 5b. In the lower levels, maximum C_{CC} values will result from cumulated differences between compression forces P_{EB} and tension forces P_{BRB} (see Fig. 5c). Examination of time history responses provided insight on brace force patterns producing the maximum force C_{CC} at a given level, and simplified brace force distributions could be proposed for design purposes. Figure 5a illustrates the distribution applicable for $C_{CC,i}$ at a level i greater than $3n/4$, where n is the number of floors. The multiplier γ , which varies between 0.0 and 1.0, is applied to the P_{BRB} and P_{EB} values to represent the variation in the brace forces in adjacent storeys. A second correction factor, α , is needed 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. Therefore, the compression force $C_{CC,i}$ can be 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 the exponent β varies linearly from 1.0 for the 24-storey building to 3.0 for the 8-storey building. 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 n or when n is increased. The 50th or 84th percentile estimates of C_{CC} can be obtained by using the corresponding percentile values for P_{BRB} and P_{EB} . At levels below $3n/4$, the profile shown in Fig. 5b must be used with a parabolic decay function above the two levels where maximum brace forces occur. This calculation is performed from the top floor down to the building base. For the bottom part of the central column, the same, but inverted brace load patterns are applied starting from level 1 up to the roof level as illustrated in Fig. 5c. At intermediate levels, the minimum values from the two patterns are retained for design. This two-step calculation is illustrated in Fig. 6. Figure 7a compares the analytical results and the prediction from equation (1).

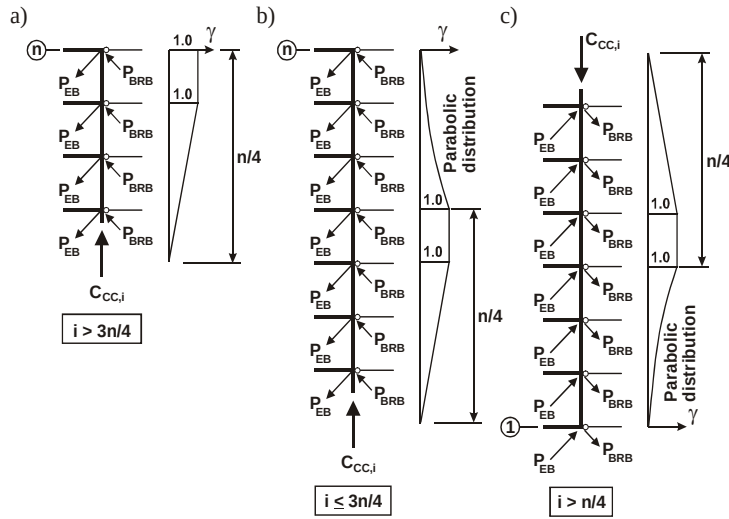


Fig. 5: Brace force patterns for determining central column forces C_{CC} in the: a) Upper levels; b) Lower levels.

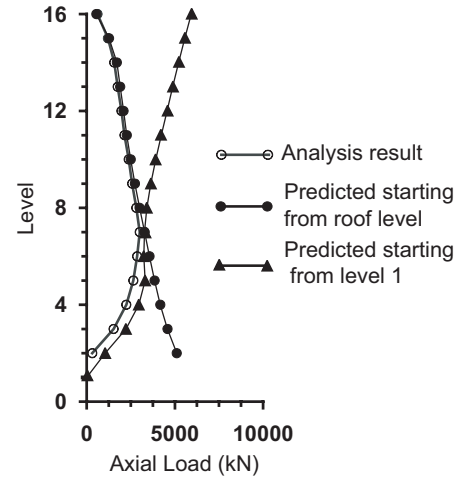


Fig. 6: Analytical (50th percentile) vs predicted C_{CC} values for the 16-story building.

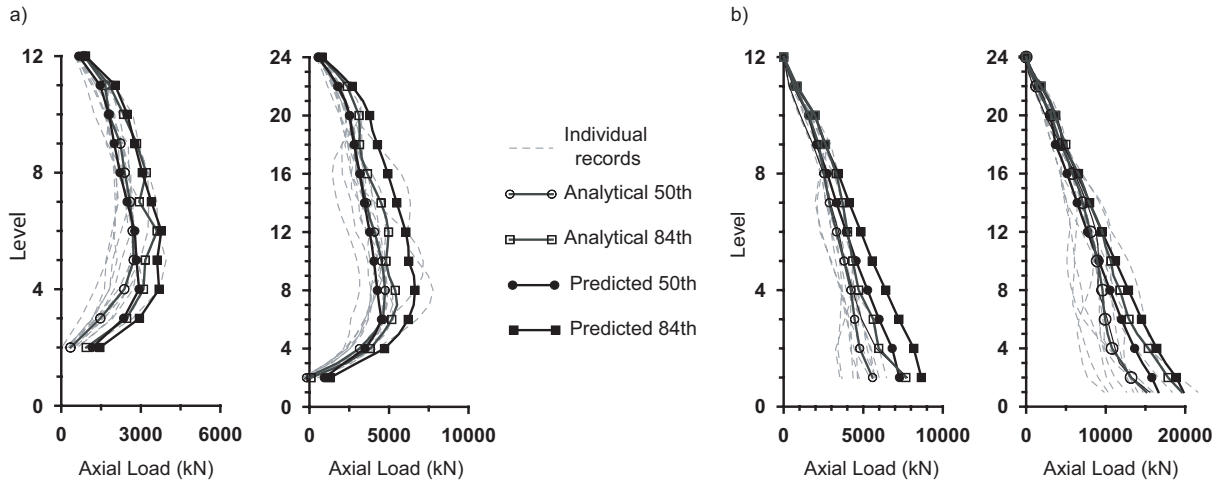


Fig. 7: Comparison between analytical and predicted values for column forces: a) C_{CC} ; b) P_{EC} .

Table 3: Mean and COV values of the analytical to predicted ratios for Dual BRB member forces.

n floors		50 th percentile				84 th percentile			
		P_{BRB}	P_{EB}	P_{EC}	C_{CC}	P_{BRB}	P_{EB}	P_{EC}	C_{CC}
8	Mean	1.02	0.80	0.76	0.81	1.06	0.83	0.82	0.88
	COV	0.02	0.16	0.10	0.24	0.04	0.17	0.15	0.11
12	Mean	1.01	0.86	0.88	0.94	1.06	0.87	0.88	0.90
	COV	0.03	0.15	0.15	0.29	0.03	0.12	0.13	0.12
16	Mean	0.98	0.91	0.88	0.86	1.02	0.90	0.85	0.79
	COV	0.03	0.12	0.08	0.21	0.03	0.09	0.06	0.22
20	Mean	0.95	0.95	0.94	1.04	0.99	0.99	0.96	0.85
	COV	0.04	0.09	0.11	0.23	0.04	0.07	0.05	0.19
24	Mean	0.92	0.90	0.97	1.02	0.98	0.94	0.97	0.81
	COV	0.07	0.07	0.17	0.07	0.05	0.09	0.05	0.06

For the edge columns, seismic induced compression forces at a level i , $P_{EC,i}$, arises from the accumulation of the vertical components of the elastic brace axial loads developing simultaneously above the level under consideration. The same brace force vertical distribution patterns as those proposed for the central columns are also used, which gives equation (2) below. As shown in Fig. 7b and Table 3, reasonable column force estimates are obtained from this approach.

$$P_{EC,i} = \sum_{j=i}^n \gamma_j P_{EB,j} \sin \theta_j \quad (2)$$

4 CONCLUSIONS

A Dual BRB frame system is proposed to enhance the seismic stability performance of multi-storey steel structures of up to 24 storeys. The system is found to be adequate for crustal and sub-crustal earthquake hazard. Similar conclusions can be drawn for the 8- and 24-story Dual BRB frames under the subduction earthquakes, but the 12-, 16- and 20-storey frames did not display satisfactory performance under these ground motions. For these structures, the design seismic loads must be increased to obtain acceptable response. Based on the time history analysis results, expressions have been proposed to predict 50th and 84th percentile values of the axial loads in the elastic truss components of the Dual BRB framing system. Values predicted with these expressions were in good agreement with analytical results. These guidelines only apply to the structures examined in this study. Validation for structures with a different geometry or located in other seismic areas should be performed in future research.

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KEYWORDS

Multi-story buildings, braced steel frames, buckling-restrained braces, earthquakes, stability.