Effective Steel Braced Frames for Tall Building Applications in High Seismic Regions

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Abstract. Steel braced frames are commonly used for building structures in seismic active regions. However, steel braced frame systems are limited to low- and medium rise structures because they are prone to concentration of inelastic demand resulting from adverse P-Δ effects and lack of vertical stiffness continuity. The article introduces a modified inverted-V buckling braced frame configuration in which one of two bracing members at every level is replaced with a conventional brace designed to remain elastic and form with the beam member an elastic secondary system providing the system with positive post-yielding storey shear stiffness annihilating P-Δ effects upon yielding of the BRB members and ensuring stable seismic response for tall building applications. The anticipated behaviour and design approach of the proposed E-BRBF system is first described. The stability of the system is then verified through nonlinear response history analysis for 20-, 30- and 40-storey buildings subjected to ground motions from shallow crustal, subduction in-slab, and subduction interface earthquakes. The analysis results are compared to those obtained with conventional BRBFs. The comparison shows that the proposed E-BRBF system can significantly enhance the seismic response of tall buildings, with reduced and more evenly distributed peak storey drift demand over the structure height.

Keywords: Buckling restrained braces · Soft-storey response · P-delta effects · Post-yielding stiffness

1 Introduction

Steel buckling restrained braced frames (BRBFs) exhibit stable hysteretic response under cyclic inelastic lateral deformations and therefore represent a cost-effective solution for seismic applications [1–4]. Buckling restrained bracing members however display modest post-yielding stiffness, which makes BRBFs sensitive to the negative storey shear stiffness from P-Δ effects, $\Sigma P/h_s$ in Fig. 1a, on storey shear response upon brace yielding. In multi-storey buildings, this negative storey shear stiffness combines with the limited capacity of BRBFs to vertically distribute the inelastic demand over the structure height, making the structure prone to concentration of storey drifts and large residual drifts [5–7]. This undesirable response is more pronounced for BRBFs subjected to long duration ground motions from subduction earthquake that have substantial energy in the long period range.

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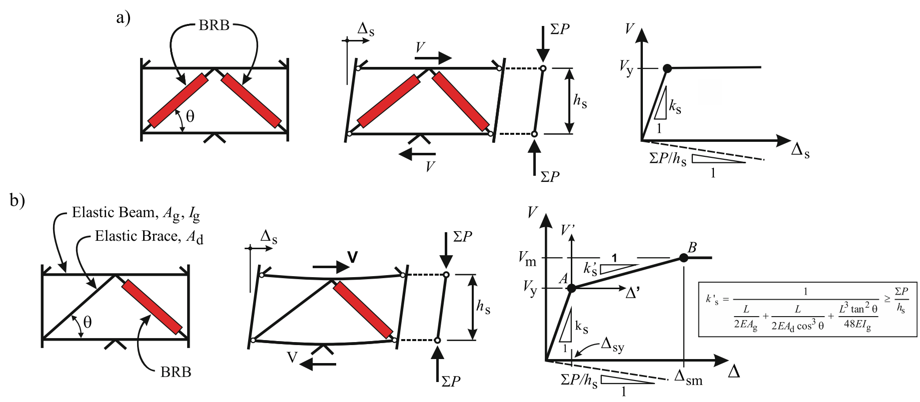

Fig. 1. Inverted-V BRBF: a) Conventional configuration; b) Proposed E-BRBF configuration with elastic braces.

Traditionally, stability effects in seismic design have been addressed in codes by amplifying the design seismic loads to compensate for the loss in storey shear resistance resulting from P-Δ effects. Studies have shown, however, that while the approach can reduce or delay the occurrence of dynamic instability, it is not sufficient to ensure stable inelastic response under seismic events. Minimum base shear requirements and building height limits that are prescribed in codes also contribute to mitigating the risk of collapse of multi-storey frames due storey drift concentration and global instability. For instance, BRBFs are limited to 40 m in Canada [8]. However, these limits represent severe restrictions for a steel braced frame system that offers a desirable stable inelastic cyclic response.

A more effective approach to mitigate stability effects on the inelastic seismic response of multi-storey BRBFs consists in incorporating a secondary structural system that can develop and maintain a storey shear stiffness larger than the negative stiffness due to P-Δ effects over the range of anticipated seismic deformations. This can be achieved with a back-up moment resisting frame acting in parallel with the BRBF, as demonstrated in [9, 10]. In V and inverted-V BRBFs, positive storey shear stiffness can also be achieved by simply replacing, at every level, one of the two bracing members with a conventional brace designed to remain elastic upon yielding of the adjacent BRB member [11]. As illustrated in Fig. 1b, the storey shear elastic stiffness k'_s is provided by the elastic brace acting in series with the elastic beam deforming in flexure. At every level, the beam is selected with sufficient flexural stiffness and strength such that k'_s exceeds $\Sigma P/h_s$ up to the anticipated storey drift and, thereby, P-Δ effects on inelastic response are annihilated. The study in [11] demonstrated that the proposed system, referred to herein as the E-BRBF system, can ensure stable inelastic response for 16-storey buildings subjected to ground motions from crustal, deep in-slab subduction, and interface subduction earthquakes, for the critical case where BRB members exhibit an elastic-perfectly plastic hysteretic response with no strain hardening. In this article, the response of the proposed E-BRBF system is compared to that of conventional BRBFs for 20-, 30- and 40-storey buildings.

2 Prototype Buildings

2.1 Buildings Studied

The prototype buildings are office buildings located on a site class C (stiff soil) in Vancouver, British Columbia, Canada. The floor plan view of the structure and design gravity loads are presented in Fig. 2. The storey heights are 4.5 m in the first storey and 4.0 m in the upper ones. As shown, the braced frames are located on the exterior column lines and the inverted-V brace configuration was adopted for all frames. For the 20-storey buildings, the braced frames are two bays in width over the full building height. For the 30- and 40-storey E-BRBF buildings, the frames are 4-bay wide in the lower storeys (segment 1) and two bay wide in the upper levels (segment 2). The same configurations are also used for the 30-storey BRBF. For the 40-storey BRBF, the frame is 6-bay wide in segment 1 and 4-bay wide in segment 2.

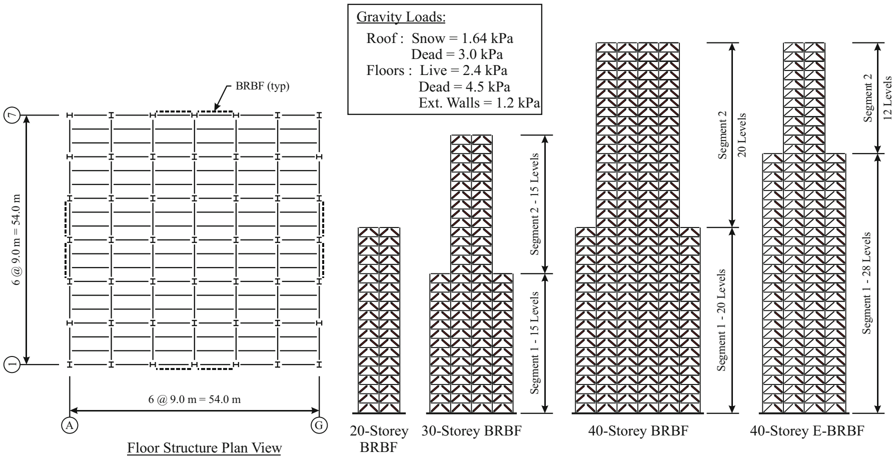

Fig. 2. Prototype building structures.

In the NBC [8], the seismic design base shear, V, is determined from:

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

where S is the design spectrum, which is defined by the site-specific 2% in 50 years UHS ordinates at periods of 0.2, 0.5, 1.0, 2.0, 5.0, and 10 s, T_a is the building fundamental period, I_E is the importance factor, M_v is a factor that accounts for higher modes on base shear, W is the building seismic weight, and R_d and R_o are respectively the ductility- and overstrength-related force modification factors. The design spectrum for the selected site is shown in Fig. 4. The period T_a can be taken equal to the period T_1 from modal analysis, except that T_a must not exceed 0.05 h_n for steel braced frames, where h_n is the building height in meters. In addition, a minimum design base shear corresponding to the value obtained with T_a = 2.0 s must be considered when the period T_a exceeds 2.0 s. For the buildings studied, both the M_v and I_E factors were taken equal to 1.0. For steel BRBFs, the R_d and R_o factors specified in the NBC are equal to 4.0 and 1.2, respectively. In the NBC, the anticipated inter-storey drifts including inelastic response must not exceed 2.5% h_s, where h_s is the storey height.

2.2 Design of the BRBF and E-BRBF Structures

For each building, the BRBFs were designed in accordance with the provisions of the NBC and the CSA S16 steel design standard [12], ignoring the NBC prescribed height limit of 40 m. For the E-BRBFs, higher values of R_d equal to 6.0 for the 20-storey and 8.0 for the 30- and 40-storey designs were used in anticipation of a more stable response resulting from the elastic post-yielding stiffness exhibited by these system. For all structures, design axial loads for the BRB members were determined by combining the loads from the concomitant gravity loads (1.0 D + 0.5 L + 0.25 S) to the seismic induced loads from response spectrum analysis (RSA). In the NBC, RSA results must be scaled up by the ratio 0.8 V/V_{RSA}, where V is the base shear from Eq. (1) and V_{RSA} is the base shear from RSA, when V_{RSA} is lower than 0.8 V. Periods T_1, T_2, and T_3, as obtained from modal analysis after completion of the designs, are given in Table 1. As shown, for all structures, the computed period T_1 and the upper limit on the period T_a, $T_{a,max}$ = 0.05 h_n, both exceeded 2.0 s and the minimum base shear obtained with S(2.0 s) = 0.255 had to be used in design.

Table 1. Structure properties.

Frame	#storeys	h_n(m)	$T_{a,max}$(s)	T_1(s)	T_2(s)	T_3(s)	V/W ()	*Tonnage*1 (t)
BRBF	20	80.5	2.03	3.41	1.20	0.68	0.0425	231
	30	120.5	3.03	4.32	1.65	0.86	0.0425	551
	40	160.5	4.03	4.80	1.78	0.98	0.0425	1119
E-BRBF	20	80.5	2.03	3.69	1.25	0.70	0.0283	280
	30	120.5	3.03	5.03	1.92	1.00	0.0212	640
	40	160.5	4.03	7.09	2.24	1.25	0.0212	1122

[1] Tonnage per braced frame, excluding the weight of the bracing members

In CSA S16–14, design seismic loads must be amplified to account for P-Δ effects on inelastic seismic response and notional horizontal loads equal to 0.5% of the gravity loads must be considered in seismic design loads. These stability provisions were considered in the design of the conventional BRBFs but ignored in the design of the E-BRBFs to verify if the proposed system could ensure a stable inelastic response without the current stability provisions. For both frame systems, capacity design principle was performed for the selection of the beams and columns assuming probable BRB resistances obtained with strain hardening and compression strength adjustment factors (ω, β) equal to 1.4 and 1.1, respectively.

For the E-BRBFs, the elastic braces and beams were chosen to develop at each level a storey shear stiffness, k'_s, equal to $\Sigma P/h_s$. Design loads for the elastic braces (axial compression load) and beams (axial compression load plus bending moment) were determined at a target shear inter-storey drift, Δ_{sm}, equal to 2% h_s so that the stiffness k'_s could be maintained up to that deformation. Beyond that shear storey drift, beams were expected to yield in flexure. This approach led to minimum E-BRBFs designs as the frames did not exhibit positive post-yielding stiffness that could provide self-centering capabilities $\left(k_s' = \Sigma P/h_s\right)$ and possessed no significant reserve against beam yielding for Δ_s beyond 0.02 h_s. The E-BRBF behaviour described below therefore reflects the minimum performance that can be expected for the system.

As shown in Table 1, the E-BRBFs and BRBFs require similar amounts of steel, but the former only requires half the number of BRB members and could therefore represent a more cost-effective solution.

3 Nonlinear Response History Analysis

3.1 Structure Models

Nonlinear response history analysis (NLRHA) of the braced frames was performed with the OpenSees computer analysis program [13]. Two-dimensional models of the braced frames were used. Beams, columns and elastic braces were modelled using the force-based nonlinear beam-column elements with fiber discretization of the cross section. The selected material was Steel02 with kinematic and isotropic material was used for these members. The BRB members were modelled using truss elements with an equivalent axial stiffness based on 1.5 times the BRB core areas to account for the higher axial stiffness at the end protrusions and connections. The Steel4 material was selected for the BRB members, with properties calibrated using an automated process that minimized the error between the predicted and measured test data [14]. Examples of calibration results are shown in Fig. 3.

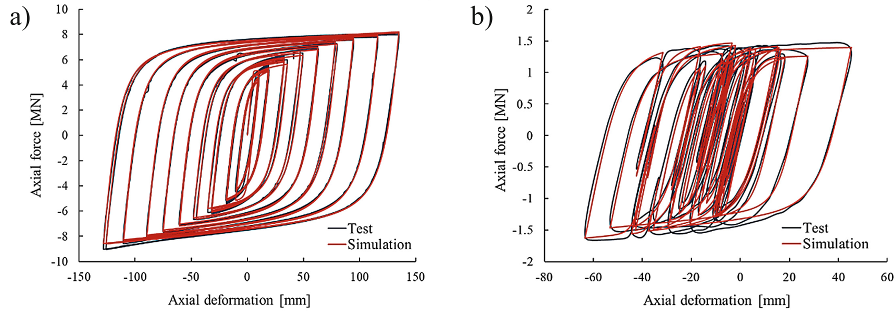

Fig. 3. Validation of the Steel4 material properties against BRB test results under: a) symmetrical cyclic loading protocols with stepwise incremented displacements; and b) seismic induced displacement protocols [14].

Prior to performing the seismic analyses, tributary concomitant gravity loads due to 1.0 D + 0.5 L + 0.5 S were applied to the frame models. The braced frame models also included a leaning P-Δ column modelled using corotational truss elements that supported the remaining of the gravity loads acting on half the building area. At each level, the lateral displacement of the leaning column node was constrained to be same as that of the node at mid-width of the brace frame, which allowed capturing the axial load demand in the braced frame beams. Large displacement analysis with corotational geometric transformation was performed. Rayleigh damping based on tangent stiffness corresponding to 3% of critical in modes 1 and 3 was assigned to the model.

3.2 Ground Motion Time Histories

The seismic hazard in Southwest British Columbia is contributed by shallow crustal, deep in-slab and interface subduction earthquakes. As per NBC, the structures were subjected to an ensemble ground motions records consisting of three suites of 11 records for each type of earthquakes. For each structure, the ground motions of each suite were scaled to match the design spectrum over suite specific period ranges that covered a period range spanning from 0.15 to 2.0 T_1 of the building. The 5% damped spectra of the scaled ground motions for the 20-storey BRBF are plotted in Fig. 4.

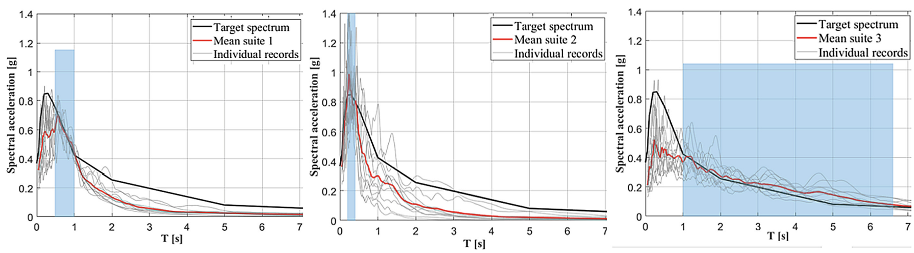

Fig. 4. 5% damped acceleration spectra of the ground motions selected and scaled for the 20-storey BRBF structure.

4 Analysis Results

Peak inter-storey drift ratios from NLRHA for the BRBFs and E-BRBFs are presented in Fig. 5. In spite of the lower R_d factor (4.0) used in design, the BRBFs consistently experienced large and non uniform inter-storey drifts over the building height. For all three buildings, peak storey drifts under interface subduction earthquakes exceeded the NBC 2.5% h_s limit. No structural collapse was observed, however, suggesting that the minimum design base shear specified in the NBC can provide sufficient protection against this limit states, even if the BRBFs exceeded the code prescribed height limit. On the contrary, the E-BRBF system displayed lower and more uniformly distributed storey drifts for the three buildings. In all cases, the peak inter-storey drifts remained below the target 2% h_s value considered in design, which indicates that an R_d factor of 8.0 would be appropriate for controlling drifts for this system. Peak axial loads and bending moments observed in the beams of the E-BRBFs are presented in Fig. 6. These values are normalized with respect to the values used in design. As shown, beam forces remained below the design values under all ground motions for all beams of the three frames, which suggests that the design approach could be appropriate for this system.

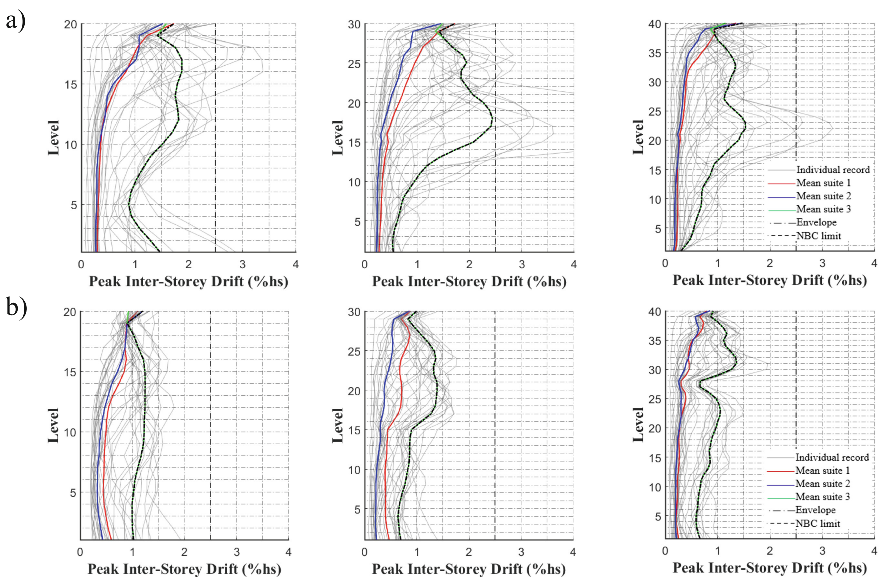

Fig. 5. Peak inter-storey drift ratios for: a) BRBFs; b) E-BRBFs.

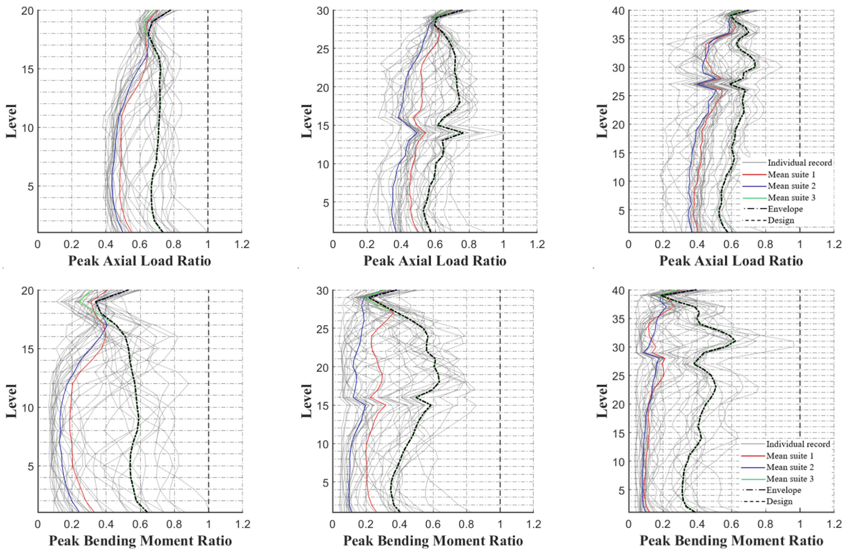

Fig. 6. Peak axial load (top) and bending moment (bottom) demands on the beams of the E-BRBFs (demands normalized to design values).

5 Conclusions

This study showed that the proposed E-BRBF system can lead to stable inelastic response for buildings located in active seismic regions that significantly exceed the height limits currently specified in building codes. Compared to the current BRBF systems, the E-BRBF system demonstrated superior inelastic seismic response under all three types of ground motions, with smaller and more evenly distributed inter-storey drifts. The system was also found to require a comparable amount of steel but with half the number of BRB members.

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