Influence of brace modelling on the seismic stability response of tall buckling restrained braced frame building structures

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Abstract

This paper examines the influence of the brace modelling on the seismic stability response of tall buckling restrained braced frames. In this approach the commonly used tri-linear model based on the backbone curves obtained from cyclic test data will be validated against calibrated models of Steel4 material in OpenSees. The validation involves evaluating the seismic response of 6-, 10-, 16- and 20-storey BRB frames using nonlinear pushover and response history analyses. These prototype structures are office buildings located in Vancouver, BC, Canada. This location was selected because the structures are exposed to three sources of earthquakes: crustal, deep in-slab subduction, and interface subduction earthquakes. Buildings in both models (tri-linear and Steel4) are analysed using OpenSees where the evaluation involves 33 scaled ground motions representative of the sources of earthquakes. For the Steel4 model, an enhanced optimisation algorithm is used to calibrate the 20 parameters and obtain high-level correlation with past cyclic tests data. Displacements, storey drifts, and force demands are evaluated for all prototype buildings for both BRB models. It is found that the tri-linear model mitigated the negative stiffness due to P-Delta effects, resulting in storey-drifts within the design limit (2.5%). On the contrary, buildings with the Steel4 BRB model experienced excessive drifts, especially under the interface subduction earthquakes.

Keywords: Buckling restrained brace (BRB); Backbone curves; Seismic stability; Nonlinear response history analysis;

1. Introduction

Buckling restrained braces (BRBs) generally demonstrate a favourable seismic performance comparing to the conventional braces [1]. The no-global buckling behaviour in compression allows BRBs to dissipate energy reliably and uniformly, especially under the design-level story drift ratios [2]. However, BRBs demonstrate modest strain hardening which makes this choice of lateral load resisting system vulnerable against P-delta effects, especially in tall buildings subjected to long duration subduction earthquakes [3]. Since the strain hardening of BRBs is a complex combination of several factors [4], predicting it mathematically may not be possible. Therefore, simplified numerical models (e.g. tri-linear model, bi-linear model) are developed using cyclic test data to simulate the hysteretic response of BRBs. The most common simplified model is the tri-linear, where it has been used in various studies especially for design purposes e.g. [5-7]. In these simplified models, the elastic and inelastic responses of BRB are simulated either by using three stiffness slopes (tri-linear models Fig. 1a), or by using two slopes (bi-linear models Fig. 1b). These slopes are mainly governed by the area and the yielding stress of the steel core in addition to the strain hardening and compression strength adjustment factors, ω and β, respectively. These factors, in turn, vary depending on the type of BRB. This study will cover the most common two types of BRBs, i.e. BRBs exhibiting **L**ow post-yielding **H**ardening response (**LH**) and BRBs displaying **H**igh post-yielding **H**ardening response (**HH**).

In this article, the structural response of 6-, 10-, 16-, and 20-storey prototype buildings will be evaluated using both types of BRB (LH and HH), firstly, using the classical tri-linear representation of BRBs of Fig. 1a adopted by Dutta and Hamburger [5] and, secondly, using the recently developed Steel4 material of Zsarnóczay [8]. The evaluation will be mainly conducted using OpenSees platform [9], where the modeling parameters of the tri-linear representation are obtained from previous cyclic test data for both types of BRBs. Same cyclic test data will be used to calibrate the 20 parameters of the Steel4 material using an enhanced multi-level automated process to achieve highly matched results. This paper will also spot a light on the importance of accounting for the geometrical nonlinearity of beams during design, as this aspect is currently neglected in design codes and appeared to have an impact on the stability of BRB frames.

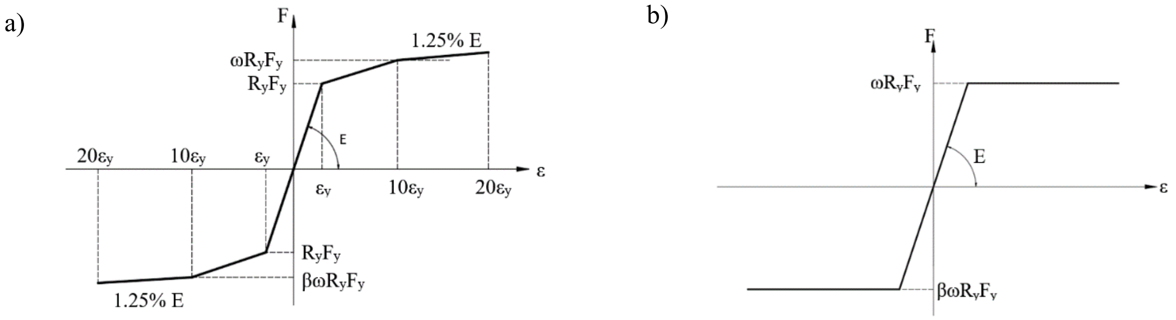

Fig. 1 – Simplified BRB material models. a) Tri-linear BRB model adapted from [5]; b) Bi-linear BRB model

2. Calibration process

2.1. Backbone (tri-linear and bi-linear) models

Modelling parameters for the tri-linear and bi-linear models for the LH BRBs were obtained from cyclic test data generated by Dehghani and Tremblay [10] on all-steel BRB members. For the HH braces, values of ω and β published by Newell et al. [12] from cyclic tests on concrete filled tube BRBs were used. The backbone modelling parameters from these past test programs and used in this study are provided in Table 1.

Table 1 – Properties of tested BRBs

BRB type	Specimen ID	Backbone parameters	
		$\omega_{av\ last\ cycle}$	$\beta_{av\ last\ cycle}$
LH	S4, S7	1.4	1.1
HH	3G, 4G	1.58	1.04

2.2. Steel4 model

The values of the parameters required for the OpenSees Steel4 material were obtained from the same test data using a calibrating algorithm initially proposed by Dehghani [11] and enhanced for this study. The calibration process uses a MATLAB function (*fminsearch*) to minimize the error between the test data and the OpenSees simulation results. In the enhanced algorithm , the error is calculated by interpolating the simulation ordinates at abscissa points of the test cycle-by-cycle. Since it is an iterative approach, it allows re-calibrating parameters depending parameters calibrated in a previous cycle (multi-level calibration) until error tolerance or the maximum number of iterations is reached. This allows achieving highly matching results with the test data, especially when using OpenSees Steel4 material, given its high adaptivity. The tested specimens for the two types of braces (LH and HH) varied in core-lengths, core-area, and yielding stress. Table 2 summarises the properties of the tested specimens considered.

Table 2 – Properties of tested BRBs

Type	Specimen ID	Restraining system	L_{core} [mm]	A_{core} [mm^2]	Tested yielding stress [MPa]
LH	S4, S7	Steel	3000	2857	385
HH	1G	Concrete	3365	7741	258
	3G, 4G	Concrete	3668	17419	258

After calibration, it was noticed that Steel4 material was able to simulate the quasi-static response with a high accuracy, both in compression and tension (Fig. 2). Additionally, the calibration parameters of Steel4 material were validated using quasi-static seismic tests performed previously on Specimen S7 in [10] and a good agreement was also obtained under this loading protocol (Fig. 3).

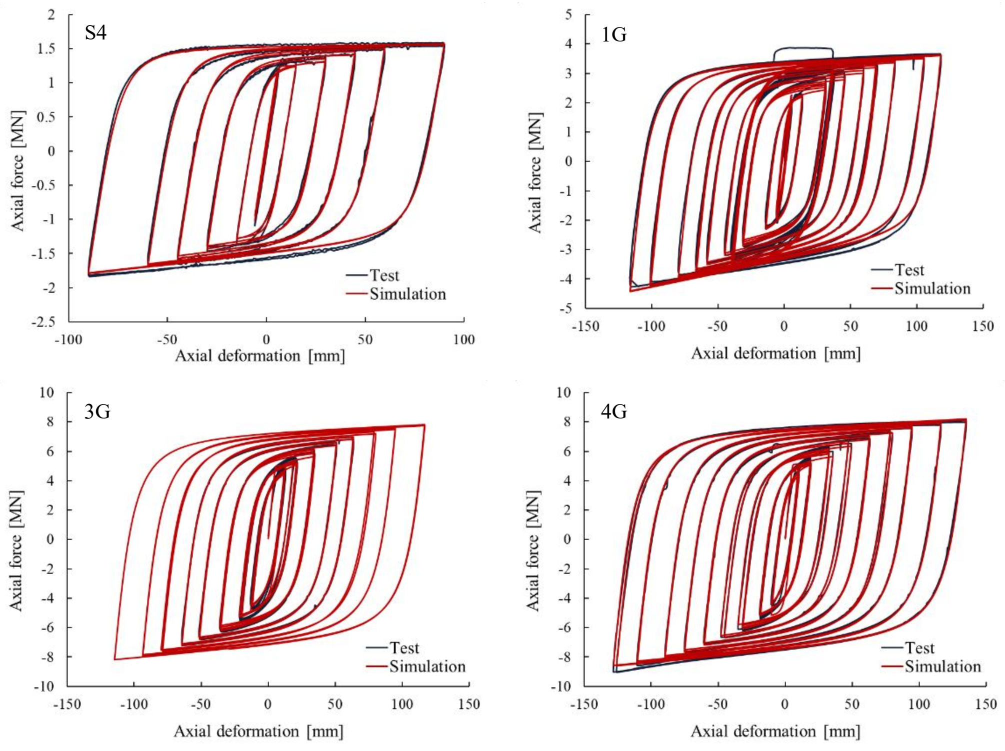

Fig. 2 – BRB Steel4 model calibration

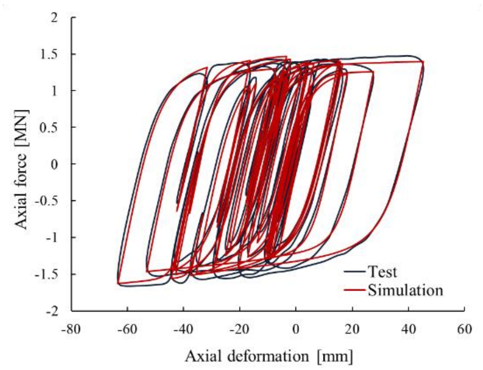

Fig. 3 – Calibration verification using seismic signal (S7 specimen)

3. Building Description and Modelling

3.1. Prototype building description and design parameters

Four office buildings having 6, 10, 16, and 20-storeys, assumed to be located on a class C (firm ground) site in Vancouver, British Columbia, Canada, were used to examine the influence of BRB modelling. The reason for choosing Vancouver, is because structures are exposed to three sources of earthquakes: crustal, deep in-slab subduction, and interface subduction earthquakes. Buildings had tall first floors (4.5 m) with no basements and with uniform storey heights (4 m). The structures were designed in accordance with the 2015 NBC [13]. NBC prescribes a height limit of 40 m for buildings with BRB frames in high seismic zones such as Vancouver. Therefore, the number of storeys in these buildings were chosen to be either within the code height limit (6-storey), at the edge of the limit (10-storey) or beyond the limit (16- and 20-storey). Two-bays braced frames with chevron bracing configuration were adapted for the 20-storey building, while only single-bay frames with the same bracing configurations were used in the three left buildings. The braced frame columns were strong-axis oriented to achieve a safer collapse margin ratio [14], while the gravity columns were oriented given their architectural layout. A constant bay width of 9 m was generalized among the four buildings in both directions. Floor layouts, loads, gravity columns and bracing configurations are given in Fig. 4.

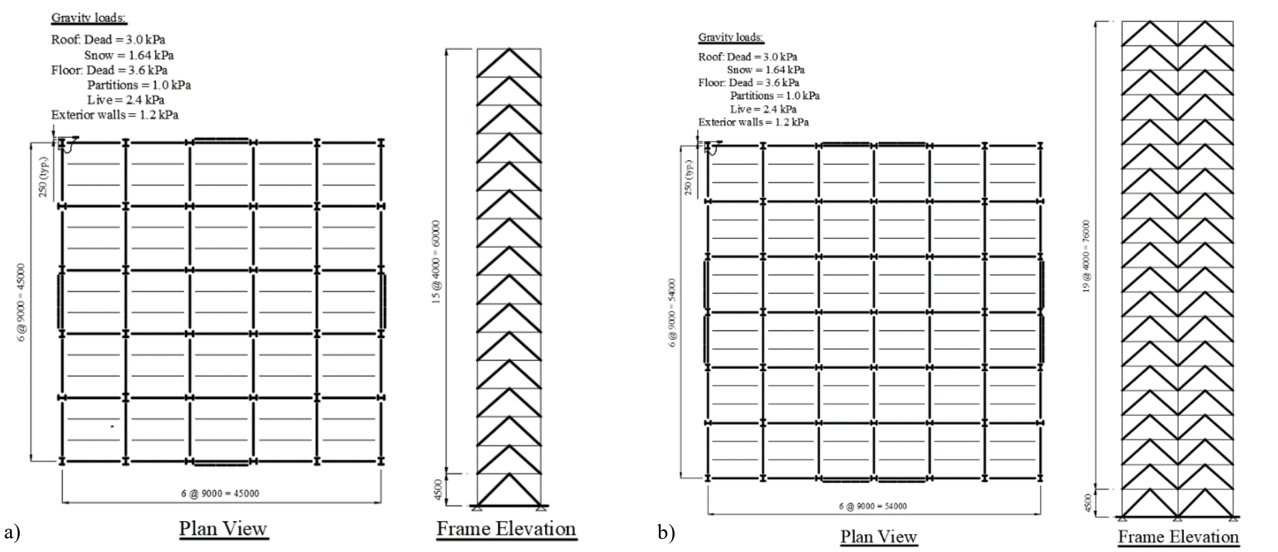

Fig. 4 – Prototype building layouts for: a) 16-storey building; b) 20-storey building

For all buildings, design seismic loads were determined using ductility-related force modification factor (R_d) of 4 and an overstrength related factor (R_o) of 1.2. Forces in BRB members were obtained from response spectrum analysis. Given the low spectral acceleration in case of the 16- and 20-storey buildings, their base shear was governed by the minimum design limit. Therefore, these two buildings were designed for seismic demands larger than their spectral acceleration demands (Fig. 5).

The BRB frames were designed following the provisions of the CSA S16 steel design standard [15]. Two designs per building were performed to accommodate the use of LH and HH BRBs because of the differences in the adjusted resistances of the braces that had to be considered for the design of the beams and columns. For both LH and HH cases, the strain hardening adjustment factor (ω) and the compression strength adjustment factor (β) were taken equal to the values given in Table 1. Braced frame beams were designed as beam-column members with a single out-of-plane lateral support at mid-span. Regarding the in-plane direction, it was assumed that one BRB will be in compression and can provide the in-plane stability requirements for the beams at mid-span. Columns were considered laterally supported at floor levels in both directions (full length columns).

Ground motions were selected and scaled with accordance to method A given in NBC [13], where the design spectrum of Vancouver was divided into three main scenario-specific period ranges T_{RS}. Each period range T_{RS} was based on the dominant periods of the ground motions associated with the magnitude-distance scenarios dominating the hazard for each of the three sources of earthquakes contributing to the hazard at the site: short periods for shallow crustal earthquakes, long periods for subduction interface earthquakes and short to intermediate periods for in-slab subduction earthquakes (Fig. 5). Three suites of 11 representative ground motions were created, one for each source of earthquakes, to form a total an ensemble of 33 ground motions. In order to suit the range of fundamental periods of the four buildings, two separate ensembles of 33 ground motions were produced (**I** and **II**). For the first one, the motions were selected and scaled considering a fundamental period of 3.4 s (16- and 20-storey buildings) whereas the second ensemble was assembled assuming a fundamental period of 2 s for the lower buildings. Since no spectral matching techniques were used (i.e. frequency or period domain scaling), NBC allows a single unaccepted response among the ground motion ensemble. Further details regarding scaling with accordance to NBC can be found in [16].

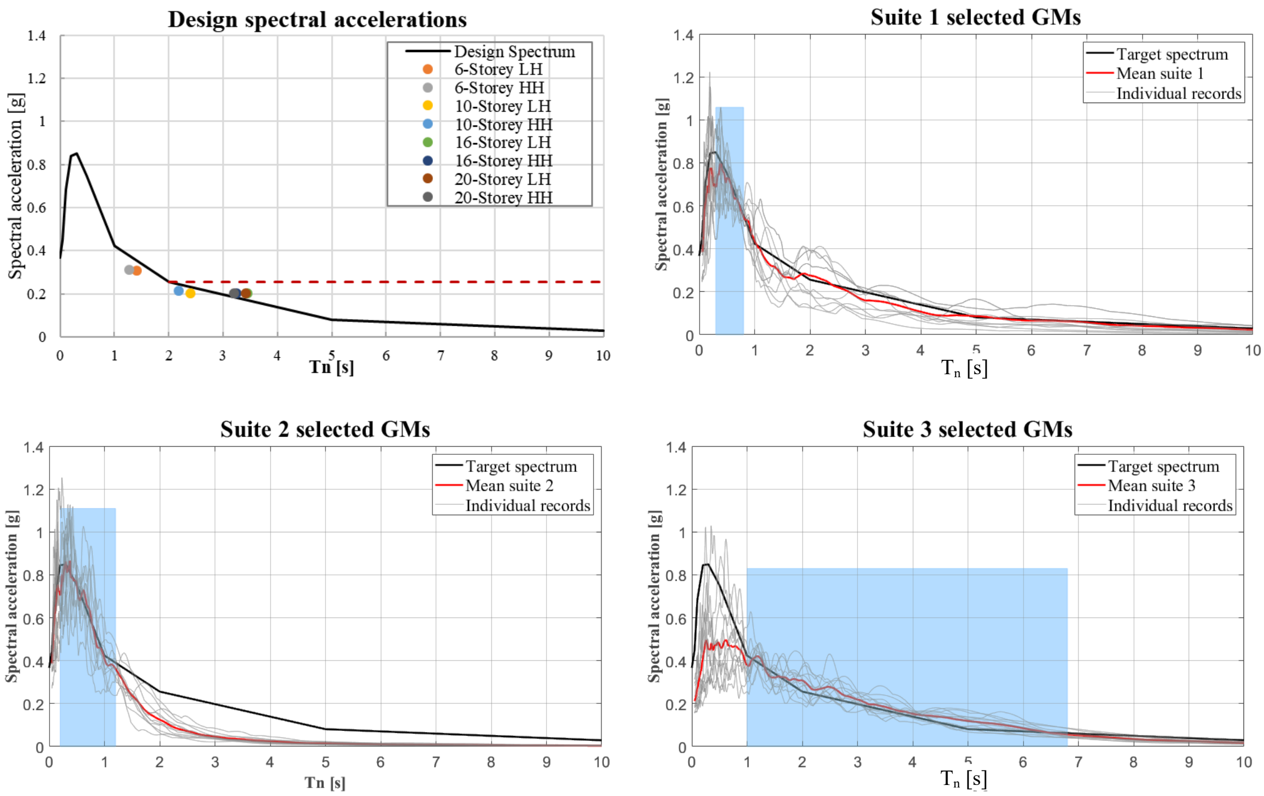

Fig. 5 – Scaled/selected ground motions (Package I)

3.2. Modelling

Two nonlinear OpenSees frame models were created for each prototype building considering both BRB types (LH and HH). In addition, BRBs for each design were modelled using both the tri-linear representation and the Steel4 material. Fig. 1a demonstrates the backbone curve of the adapted tri-linear model. Additionally, a BRB representation that uses a further simplification (bi-linear model) was also considered (Fig. 1b).

Since it is a common process to use CSI Perform3D when adopting a tri-linear modelling, an essential validation step was conducted to verify the tri-linear modelling in OpenSees. Although each program handles the material nonlinearity for elements using different techniques, both programs gave the exact same responses, as shown in Fig. 6. Given this similarity and the distributed plasticity theory adapted in OpenSees, this program was selected to run the subsequent analyses with the three BRB models (Steel4, tri-linear, and bi-linear).

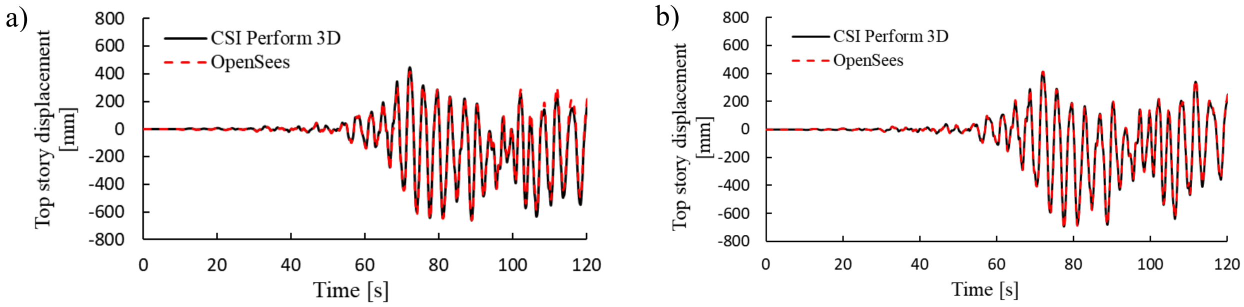

Fig. 6 – Top storey displacement responses from OpenSees vs CSI Perform 3D: a) 16-storey building; b) 20-storey building

Beams and columns of the braced frame were modelled using nonlinear beam-column elements with corotational geometrical transformation. Residual stresses using linear distribution as recommended by Lamarche and Tremblay [17] were included when modelling fiber sections in OpenSees. P-Delta effects were considered using gravity columns constrained with the braced frame at every level using rigid diaphragms. Since the gravity columns contribute to the lateral stiffness, their nonlinear behaviour was also accounted for, given their cross-sections and orientations. A pin connection located at ≈1200 mm every two storeys was assumed to simulate the splicing for all gravity columns. The columns of the braced frame in the perpendicular direction were assumed to be continuous. BRBs were modelled using truss elements with Hysteretic and Multilinear materials for the backbone models (tri-linear and bi-linear), and Steel4 material for the Steel4 models. An equivalent BRB core area was used to simulate the length of the BRB end connections and protrusions as full length BRBs were modelled [18]. Pinned connections between BRB end nodes and the connecting beam nodes were assigned. Damping was modeled using Rayleigh model with a critical damping of 3%, where the number of modes were selected to achieve 97% of mass participation factor [19]. Both mass and stiffness matrixes in the damping model were constant and reflect the initial status of the buildings [20].

4. Analysis results

4.1. Pushover

Static nonlinear pushover analysis with a simple triangular load distribution was conducted on both designs (LH and HH) of the four prototype buildings considering the BRB (tri-linear and Steel4 modelling approaches). Pushover curves and final deformation states at collapse are presented in Fig. 7. In this figure, the maximum lateral deformations shown represent the point beyond which convergence cannot be reached anymore in the analysis.

For all the analyzed buildings, examination of the pushover curves reveals a deficiency in the tri-linear model to simulate the correct static behaviour. Since pushover load is not cyclic in nature, the expected hardening developed in the BRBs must therefore be limited to kinematic hardening. The tri-linear model for which the second slope was set to reproduce both the kinematic and isotropic hardening response of BRBs subjected to cyclic loading overestimates the BRB resistance under the monotonic loading imposed in the pushover analysis. As a result, the ultimate lateral strength and lateral deformation capacity of the frames are overestimated compared to the frames with BRBs modelled using the Steel4 material. The frames exhibit a negative stiffness due to P-delta effects only when the third segment with limited stiffness (1.25% E) of the tri-linear model is reached. With Steel4, the negative stiffness due to P-delta effects occurs soon after brace yielding occurs because the BRB developed limited or no yielding plateau, given the modest kinematic hardening specified in the Steel4 model. It is also noticed that the tri-linear models lead to larger frame ductility compared to Steel4. Additionally, the final state deformed shapes of the tri-linear models exhibit more uniform storey drifts along the structure height at collapse compared to the Steel4 models, even though they were determined at larger deformations. This is attributed to the greater ability of the tri-linear model to redistribute storey drifts over the structure height and, thereby, prevent soft-storey response.

Although the objective of this study was not to compare the performance of buildings designed with LH and HH BRB members, it can be seen in Fig. 7 that, when Steel4 is used, the frames with LH BRBs demonstrate a slightly better ductile frame response compared to the same frames with HH BRBs. This is attributed to the stiffer beams present in the LH BRB frames, as these beams has to be designed for higher flexural demands due to the larger compression strength adjustment factor β considered for the LH BRBs. Contrarily, when the tri-linear model is used, the frames with HH BRBs show a better performance because of the larger strain hardening adjustment factor ω for the HH braces.

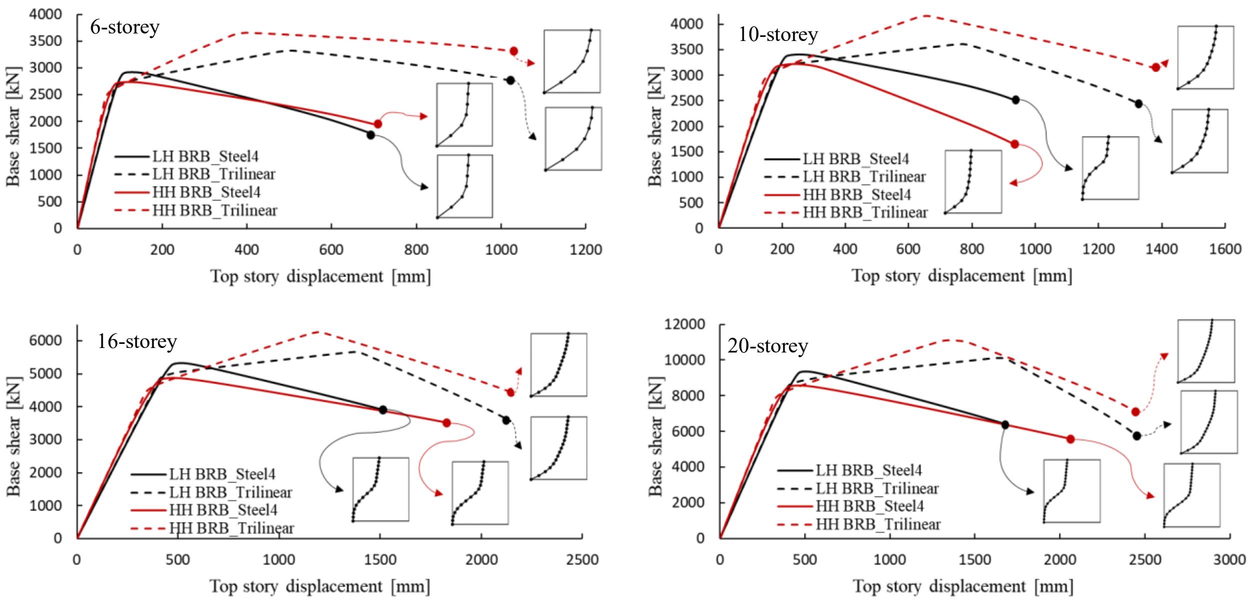

Fig. 7 – Pushover curves

4.2. Response history analysis

Nonlinear response history analysis was conducted using ground motion ensembles defined earlier. Peak storey drifts are presented for each building using the tri-linear and Steel4 models and for both BRB types in Figs. 8 and 9. For tri-linear models, all storey drift ratios are within the code limit ($\leq$ 2.5%), while excessive storey drifts were obtained when the Steel4 model was used. This can be explained by the high stiffness of the second segment of the tri-linear BRB model which contributed to control storey drifts and prevent instability failure. The analyses were also conducted with the bi-linear BRB model of Fig. 1b. The lack of strain hardening in that model caused the collapse of all frames under all long duration ground motions from subduction interface earthquakes (Suite 3). Additionally, the results with the Steel4 model revealed that the storey drifts were mainly governed by the subduction interface earthquake ground motions. The larger response of the taller buildings with longer periods and supporting larger gravity loads under long duration ground motions with longer dominant periods could be expected, especially when the BRB are modelled with Steel4 material exhibiting low kinematic hardening. As also anticipated, the frames with LH BRBs generally sustained larger storey drifts. It is important to mention that excessive drifts were obtained in the 6- and 10-storey buildings with LH and HH type BRBs, even if these buildings satisfy the building height limit prescribed in the NBC.

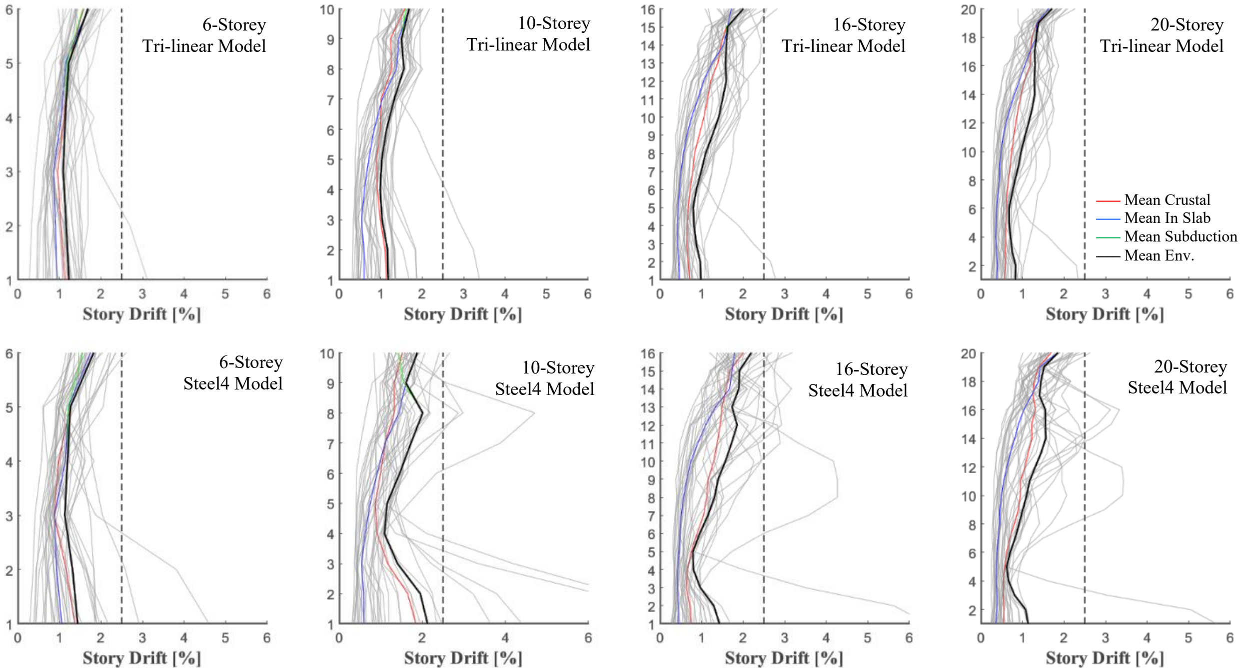

Fig. 8 – Storey drift ratios (LH BRBs)

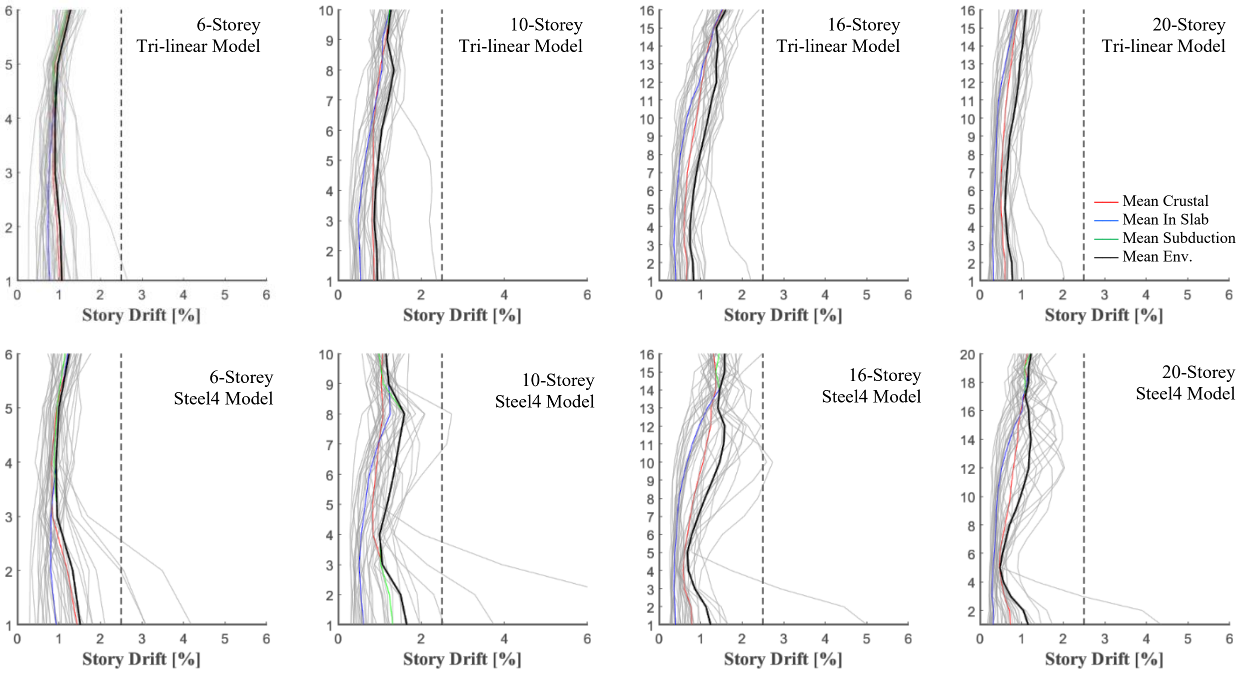

Fig. 9 – Storey drift ratios (HH BRBs)

To further investigate the BRB frame response under the subduction interface ground motions, the hysteretic response of the BRB members in the first floor of the 6- and 10-storey buildings are presented in Fig. 10 when using the tri-linear and Steel4 models. Although the ground motion excitation imposes numerous loading cycles, the number and amplitude of the cycles were not sufficient to activate significant isotropic hardening in the Steel4 model compared to the larger kinematic hardening that developed in the trilinear model under the small deformation range (< 50 mm) experienced by the brace. As shown, the BRB modelled with Steel4 is more vulnerable to large inelastic excursions. The stable response of the frame with the tri-linear BBR model is attributed to the beneficial fictitious post-yielding stiffness of that model.

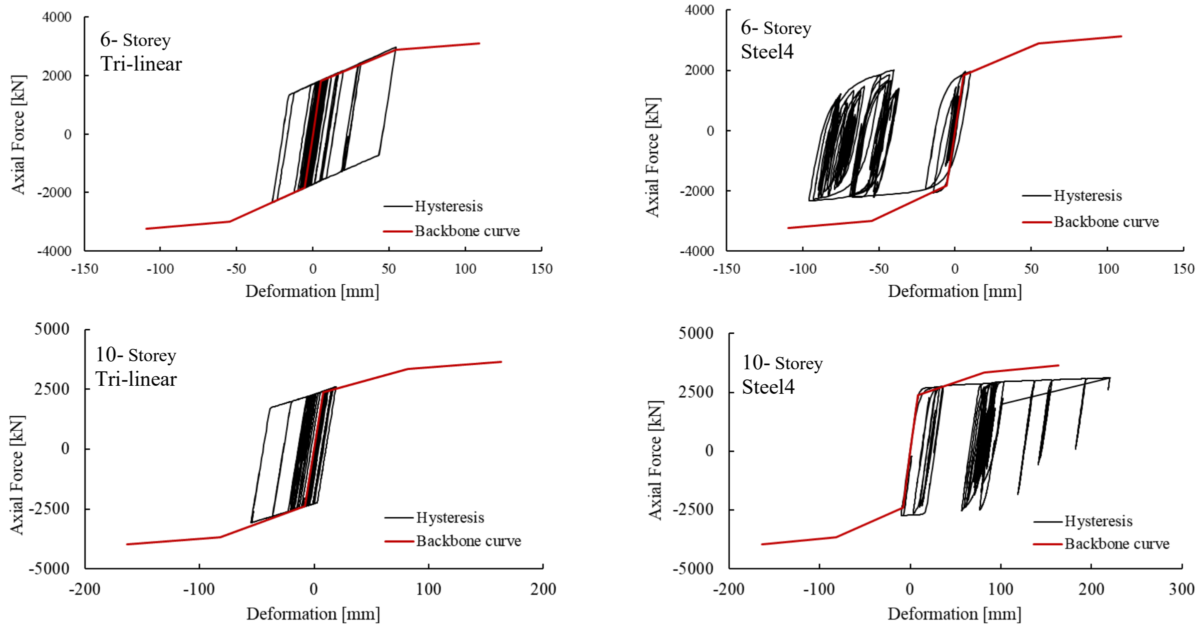

Fig. 10 – BRB hysteresis under subduction interface earthquake

Peak force demands on beams and columns are presented for both BRB models in Fig. 11 to 13. All force demands are within the design ranges for the tri-linear and Steel4 BRB models and for both types of BRBs, except for some beams in the frames with type HH BRBs. The reason for this excessive demand cases is described in the next section. For the beams, higher demand-to-capacity ratios are obtained when the Steel4 BRB model is used, and the ratios are generally larger when type HH BRBs are used.

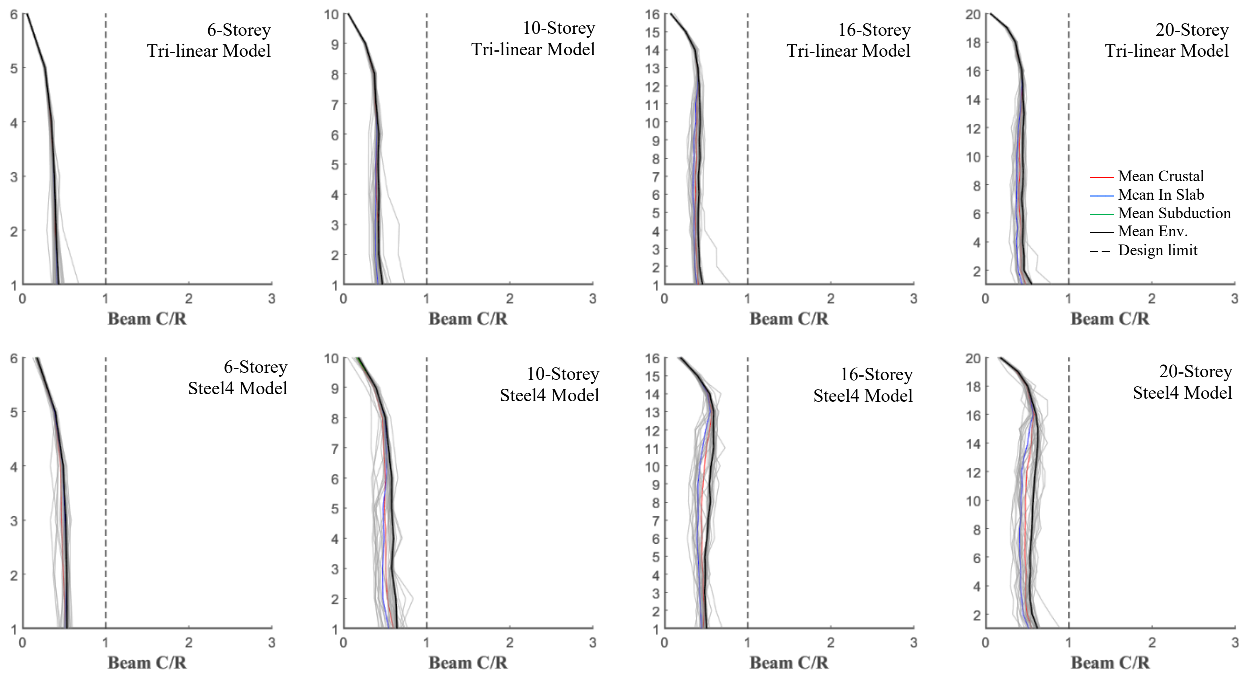

Fig. 11 – Beam demand/design capacity ratio (LH BRBs)

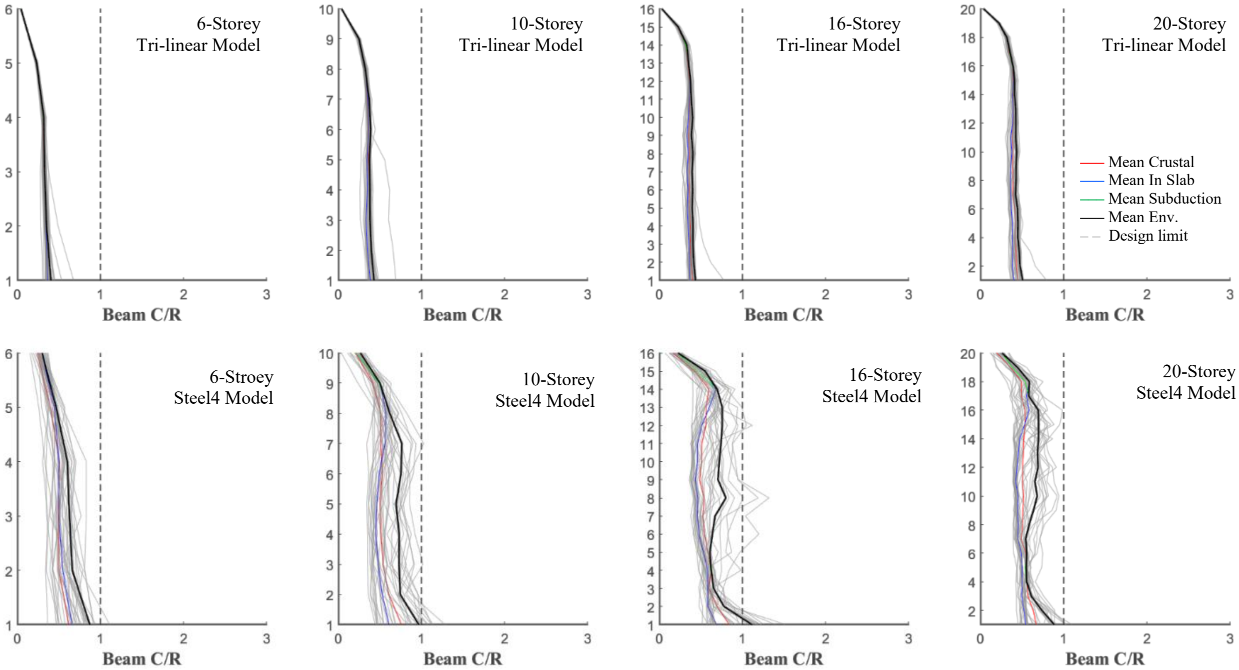

Fig. 12 – Beam demand/design capacity ratio (HH BRBs)

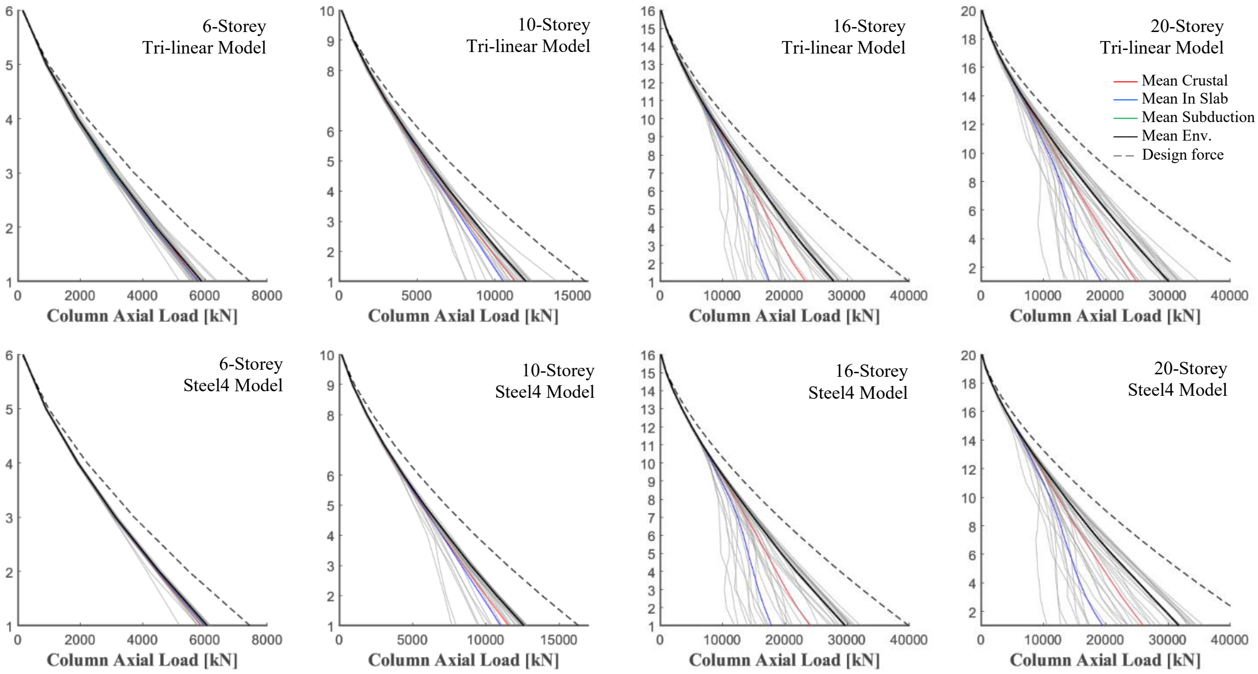

Fig. 13 – Column force demands (LH BRBs)

4.3. The effect of geometrical nonlinearity

It was noticed that beam demands slightly exceeded the design criteria when the Steel4 model was used in frames constructed with HH BRBs (Fig. 12). Close examination of the results showed that this higher than expected demand was caused by additional vertical forces being imposed on the beams in the deformed configuration. The neglected higher demands on beams due to geometrical nonlinearities. Fig. 14a illustrates the vertical force V_1 due to the difference between brace compression force (βT) and tension force (T). That force V_1 thus varies with the compression strength adjustment factor β and the BRB angle α_1. As the braced frame laterally deforms, the BRB angle changes from α_1 to α_2, causing the vertical load to increase from V_1 to V_2. This vertical force difference ($V_2 - V_1$) imposes an additional force demand on the beam. Additionally, this additional force also varies with the compression strength adjustment factor β. Fig 14b plots the relation between the ratio V_2/V_1 computed at a storey drift of 2.5% as a function of the adjustment factor β given the studied frame geometry. It is noticed that for BRBs with low β, i.e. almost symmetrical tension/compression resistances, the amplification of the vertical force due to the change is geometry is larger. This, explains the force demands exceeding beam capacities in the frames with HH BRBs (low β) when modelled using Steel4.

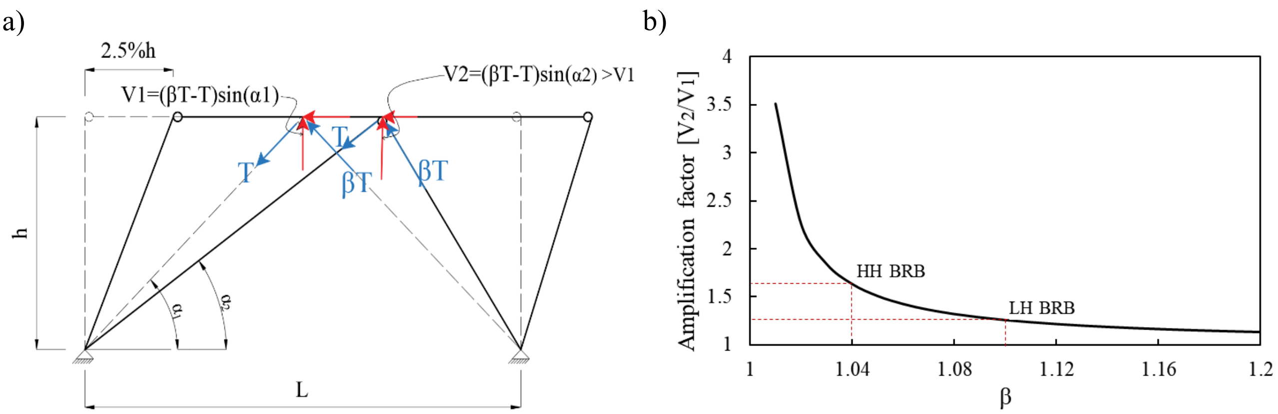

Fig. 14 – Effect of geometrical nonlinearity. a) Beam force demands; b) Amplification factor variation with β

5. Conclusions

This paper examined the influence of the brace modelling on the seismic stability response of tall buckling restrained braced frames. Four buildings with different number of storeys were modelled in OpenSees using two different BRB materials, the commonly used tri-linear material and the calibrated Steel4 material. Nonlinear static pushover was performed, followed by dynamic response history analyses conducted using 33 ground motions that represent the main three sources of earthquakes for Vancouver, BC. The main conclusions can be summarised as follows:

- The simplified tri-linear modelling overestimated the post yielding stiffness of the BRB members, preventing instability-caused excessive drifts that occurred when using the Steel4 model.
- For isotropic hardening to develop, BRBs must undergo large number of loading/unloading cycles, which may not happen before large drifts and resulting P-Delta effects occur and lead to instability failure, especially for tall buildings under the long duration ground motions from subduction interface earthquakes.
- Capacity design methodology adapted in the NBC (2015) was adequate to prevent failure of the braced frame beams and columns. However, it is recommended to design the beams using an increased compression strength adjustment factor β to account for the increased vertical loads acting on the beams due to the deformed geometry at the maximum anticipated storey drift.

6. Acknowledgments

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