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EVALUATION OF THE DEMANDS ON NON-STRUCTURAL COMPONENTS IN CONTROLLED ROCKING STEEL BRACED FRAMES

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Abstract

Controlled Rocking Steel Braced Frames (CRSBFs) are being developed as a high-performance seismic force resisting system to reduce structural damage and residual drifts. CRSBFs replace the braced bays in a typical steel structure with a braced frame that is intentionally allowed to uplift and rock during a large seismic event. As a CRSBF displaces laterally, the system stiffness is reduced due to uplift of the frame, rather than yielding, enabling the system to self-centre with minimal residual drifts. This rocking response is controlled using a combination of post-tensioning and supplemental energy dissipation. This design approach is in contrast to conventional codified steel systems, in which the earthquake forces are reduced by allowing certain steel members in the seismic force resisting system to yield, leading to structural damage and residual drifts that may render the structure economically unfeasible to repair. Extensive research has been conducted to show that CRSBFs can improve structural performance, but it is also important for designers to know if utilizing a CRSBF will provide increased, decreased, or similar non-structural component performance when comparing to more traditional yielding lateral force resisting systems.

As such, the purpose of this paper is to investigate how the demands on acceleration-sensitive attached non-structural components in buildings with CRSBFs compare to those demands with a buckling-restrained braced frame (BRBF) for the lateral force resisting system. A three-story building is designed to meet the structural drift and collapse criteria for seismic design set out by ASCE 7, twice using a CRSBF with different levels of energy dissipation, and again using a BRBF as the seismic force resisting system. Both structures are modelled in OpenSees, and nonlinear time-history analyses are performed on the three lateral force resisting systems for a suite of ground motions. Using a cascading analysis approach, these floor responses are used to compute floor pseudo-acceleration spectra to examine the demands on acceleration-sensitive attached non-structural components. The results indicate that providing a nonlinear structural response only through rocking at the base, while producing the intended benefit of elastic response of structural members, also has an unintended consequence of relatively higher floor pseudo-acceleration spectra at short periods. These demands do not appear to be caused by impact of the CRSBF on its foundation, but rather by vibration of the CRSBF in its higher modes. They are not effectively reduced by supplemental energy dissipation at the base rocking joint, but could potentially be reduced by providing multiple mechanisms for nonlinear response.

Keywords: controlled rocking frames, buckling restrained braced frames, floor spectra, non-structural components

1. Introduction

Controlled rocking steel braced frames (CRSBFs) are part of a new generation of self-centering seismic force resisting systems that have received global attention as earthquake engineers shift from a focus on life safety to a focus on resilience and post-earthquake recovery. CRSBFs can readily be designed to minimize or eliminate structural damage and residual drifts [1-4], unlike more conventional codified seismic force resisting systems that rely on plastic deformations in structural members to provide a nonlinear response that limits seismic forces. A CRSBF is designed to uplift from its foundation in response to lateral loads, and this uplifting behaviour is controlled using a combination of post-tensioning and supplemental energy dissipation, as shown in Fig. 1. Some CRSBFs have been designed to carry tributary gravity loads [e.g. 5-6], but the CRSBFs considered in this study are assumed to be effectively decoupled from these vertical loads by using special connection details [7-8]. In both cases, CRSBFs can be designed to fully self-center if the vertical loads that close the rocking joint are greater than the resistance provided by the supplemental energy dissipation devices.

Considering that residual drifts are a major factor in determining whether a structure can be repaired after an earthquake [9], self-centering systems are promising for avoiding the case where a structure is a total economic loss after an earthquake. At the same time, designing to minimize economic losses and promote rapid functional recovery post-earthquake requires consideration of the consequences of damage not only to structural components, but also to non-structural components [e.g. 10-12]. A few previous studies have considered non-structural components in buildings with CRSBFs. Notably, Dyanati et al. [13] found that a six-storey building with CRSBFs had better structural performance and reduced displacements compared to the same building with conventional concentrically braced frames, but also had larger demands on acceleration-sensitive components. Pollino [14] found that a three-storey building with CRSBFs had similar peaks in floor response spectra as the same building with buckling restrained braced frames (BRBFs) and that the floor spectra in the CRSBFs could be reduced by stiffening the frame or allowing members to yield. Previous work by the authors [15-16] has investigated these issues using different CRSBF designs than those considered here.Despite these advances, there is still a need for further cases to be considered because the greater range of design decisions that are available with CRSBFs relative to more conventional systems also makes it more difficult to draw generally applicable conclusions about the system.

This paper adds to the literature on floor response spectra in buildings with CRSBFs by considering a three-storey building located in Los Angeles, California. After highlighting key reasons to expect different demands on non-structural components compared to other steel framing systems, a pair of CRSBFs is designed, each with a different amount of supplemental energy dissipation, and a reference BRBF is also designed. The structural response of the systems is compared before presenting the floor acceleration spectra. Finally, the response immediately after impact while rocking is examined.

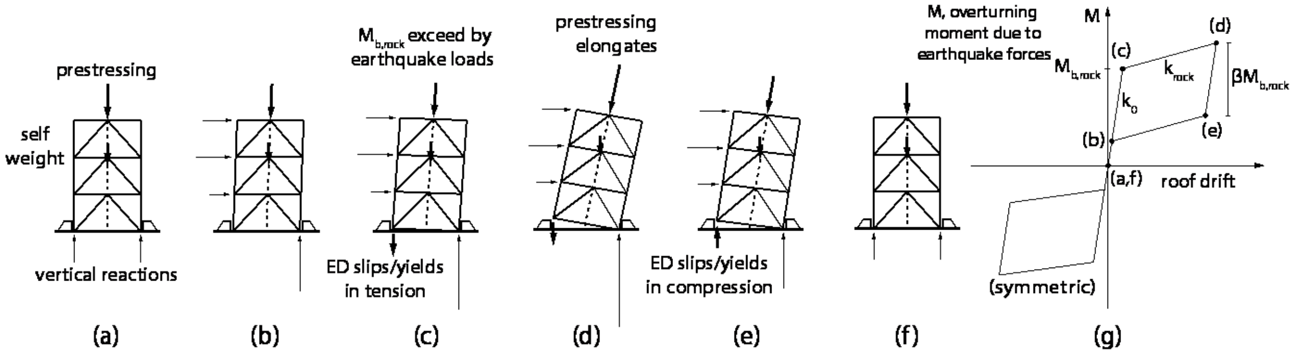

Fig. 1 – Push-pull behaviour of controlled rocking steel braced frame

2. Potential Reasons for Different Demands on Non-structural Components in Buildings with CRSBFs

In a CRSBF, all the parameters of the flag-shaped hysteresis shown in Fig. 1g can be controlled reasonably independently by the designer. The initial stiffness (k_0) is governed through the selection of frame members, the rocking moment ($M_{b,rock}$) is governed by the post-tensioning and supplemental energy dissipation, the secondary stiffness (k_{rock}) is governed by the elastic post-tensioning stiffness and the post-yielding stiffness of energy dissipation elements, and the energy dissipation parameter (β) is governed by the specified energy dissipation technology. This is in contrast to currently codified seismic force resisting systems (e.g. BRBFs), which rely on the ductility of the primary force-resisting elements to provide a nonlinear response, and therefore do not easily allow separate control of strength, initial and secondary stiffness, and energy dissipation capacity. This difference creates a need to specifically investigate the influence of these different design decisions for CRSBFs on the demands on non-structural components.

A second key difference of CRSBFs compared to codified steel systems is that the main structural elements are all designed to preclude yielding or buckling until an extreme earthquake intensity; the nonlinear behaviour of the system is designed to come primarily from the rocking response, which causes yielding only of selected energy dissipation devices. The demands on frame elements have been the subject of a great deal of research over the last decade [e.g. 1-4, 17-19], with general consensus that the demands are not effectively limited by the base rocking mechanism because of higher mode effects, and a variety of approaches to determining appropriate design forces for the frame elements to produce the desired overall performance. This is in contrast to codified systems, where at least some of the main structural elements are designed to yield in response to an earthquake. The influence of the higher modes on the elastic force demands on the main structural elements of CRSBFs could be expected to extend to the demands on acceleration-sensitive attached non-structural components.

Third, the rocking response of a CRSBF is associated with column impact during rocking, which has been suggested as a cause of severe acceleration demands [20], and with abrupt stiffness increases that have also been demonstrated to cause acceleration spikes in certain cases [21]. Some reports from the 2011 Christchurch earthquake suggest severe acceleration demands in buildings that would be expected to have such abrupt stiffness increases [20], but shake table testing of a CRSBF showed large vertical accelerations immediately above the column bases but no clear impact-induced horizontal accelerations on the seismic masses [4]. Acceleration demands that are instigated by the rocking mechanism would be unique relative to codified steel seismic force resisting systems, which do not have impact or abrupt stiffness increases.

3. Design of Seismic Force Resisting Systems

The structure considered in this study is on a site class D in the Los Angeles region, and was designed using the design spectrum defined in ASCE 7-16 [22] with MCE$_R$ spectral response acceleration parameters for short periods and at 1 s of S_{MS} = 1.5 g and S_{M1} = 1.0 g, respectively. The structural floor plan is shown in Fig. 2. The storey heights were constant at 4.57 m. The CRSBFs had a chevron configuration and were designed to be 80% of the bay width in order to fit between gravity columns, while the BRBFs had two braced bays per frame with diagonal braces. The floors and the roof had seismic weights of 10,090 kN and 6,440 kN, respectively. Both systems were designed in a manner consistent with currently available design guidance but modified slightly to achieve peak interstorey drifts of 1.5% for both designs, such that the demands on acceleration-sensitive attached non-structural components could be compared from a basis of similar demands on displacement-sensitive non-structural components. This value was chosen, even though drifts of 2% are permitted by ASCE 7-16 [22] for Risk Category II structures, because the focus is on a system that would likely be designed with the intent of better performance than the code minimum. This design process is described in more detail in the following subsections.

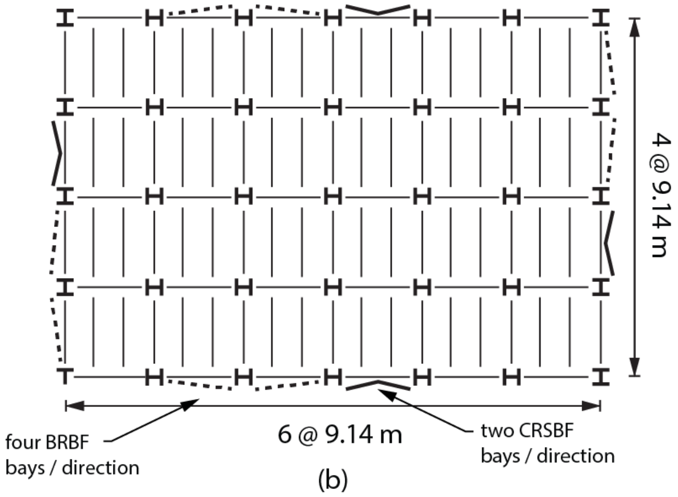

Fig. 2 – Building floor plan

2.1 Controlled Rocking Braced Frames

The CRSBFs were designed based on the design methodology described by Wiebe and Christopoulos [23]. This process requires selecting the response modification factor R and the energy dissipation ratio β. Based on an assumed first-mode period, the rocking moment is then calculated using the design spectrum, and the post-tensioning and energy dissipation devices are designed to resist this rocking moment and provide the desired level of energy dissipation. The response modification factor was initially taken as $R = 8$ [e.g. 2, 3, 24] but increased in subsequent iterations to achieve the target median peak interstorey drifts of 1.5%. Previous study [25] has shown that values of R larger than 8 can still produce acceptably low probabilities of collapse during a maximum considered earthquake. In order to assess the effect of energy dissipation on the demands on acceleration-sensitive attached non-structural components, one CRSBF was designed using $\beta = 25\%$ and another with $\beta = 90\%$. To avoid yield of the post-tensioning within the range of earthquakes considered, post-tensioning (PT) strands with an area of 140 mm^2 each and ultimate strength of 1860 MPa were designed to be located along the centreline of the frame and prestressed to 15% of ultimate; the number of strands for each design is listed in Table 1. For simplicity of design and modelling, friction energy dissipation (ED) was designed for the base of each column, with the specified slip loads provided in Table 1. After an initial estimate, the first-mode period was taken from the OpenSees [26] model described in Section 3 for subsequent design iterations, with the final values given in Table 1.

The members of the CRSBF were designed according to the dynamic capacity design procedure described by Steele and Wiebe [17]. This method takes the force effects that develop when the frame rocks to its maximum base rotation under loading consistent with the first mode, and adds the force effects that are expected to develop in the higher modes while the frame rocks. These higher-mode forces are calculated based on response spectrum analysis of only the higher modes, using a linear elastic model with boundary conditions that are modified to represent the constraints on the frame while it is rocking. The resulting higher-mode periods ($T_{2,rock}$ and $T_{3,rock}$) are compared to the higher-mode periods of the fixed base model (T_2 and T_3) in Table 1. For this stage of the design, the design spectrum was doubled to calculate the higher-mode forces so that member yielding or buckling would be rare at the design earthquake, even though Steele and Wiebe [27] demonstrated that a less conservative approach can still produce an acceptably low probability of collapse.

2.2 Buckling Restrained Braced Frame

The BRBF was designed in accordance with ASCE 7-16 [22] and AISC 341-16 [28]. In the initial design, R was taken as 8, and the fundamental period was estimated based on the approximate fundamental period provided for BRBFs in ASCE 7-16 [22]. The BRBs were designed using an assumed yield strength of 287 MPa. They were assumed to be pin connected to gusset plates at the beam-column connection of each braced bay, with a stiffness of 70% of that calculated based on the core area to account for the larger area in the connection regions. Using the model described in Section 3, this was found to result in peak interstorey drifts that exceeded the target of 1.5%, so the braces were increased proportionally and the rest of the frame redesigned as appropriate until the target drift level was obtained. As shown in Table 1, the resulting value of R was about half of the value used for the CRSBFs, but the initial period was still longer than for the CRSBFs. This is because the BRBF members are intended to yield in the design earthquake, whereas the CRSBF members are capacity designed to avoid yielding, leading to an overall stiffer design.

Table 1 – Summary of frame designs

Design	R	β	ED Force (kN)	# PT Strands	T_1 (s)	T_2 (s)	$T_{2,rock}$ (s)	T_3 (s)	$T_{3,rock}$ (s)
$\beta = 25\%$ CRSBF	10.3	0.25	222	72	0.39	0.13	0.14	0.09	0.10
$\beta = 25\%$ CRSBF	13.9	0.90	592	33	0.48	0.16	0.18	0.12	0.13
BRBF	5.4	N/A	N/A	N/A	0.72	0.32	N/A	0.20	N/A

3. Modelling of Seismic Force Resisting Systems

Figure 3a shows a schematic of the model for the CRSBF that was developed in OpenSees [26] based on models developed previously by Steele and Wiebe [17]. Elastic beam-column elements were used for all members of the frame because they had been designed to avoid yielding or buckling under design level earthquakes, as discussed in Section 2.1. Rigid elements were used to model the stiffness of the connection regions. Uplift was permitted using vertical elastic-no tension elements that were essentially rigid in compression, and sliding was prevent using similar elements oriented horizontally. A corotational truss element with an initial strain was used for the prestressing, applying a multi-linear material model with an initial stiffness of 201 GPa, a yield stress of 1670 MPa, and an ultimate stress of 1860 MPa at a strain of 0.013. This element was connected to the foundation using an elastic-no tension element in the negative direction to prevent any compressive stresses from developing when the bae rocking joint closed after yielding the post-tensioning. P-Delta effects were modelled using a leaning column that carried most of the gravity loads, assuming that special connections to transfer the lateral loads to the CRSBF could be modelled by slaving the horizontal degree of freedom of the centre node of the CRSBF to that of the leaning column at each level. Inherent energy dissipation was modelled using tangent stiffness proportional Rayleigh damping of 2% [29] in the first two modes.

Similarly, Fig. 3b shows a schematic of the model for the BRBF that was developed in OpenSees [26]. The BRBs were modelled with truss elements and the Steel02 material, calibrated as described by Buccella [30]. Elastic beam-column elements were used for the columns, with rigid offsets where the columns are stiffened by the connections, while the beams were modelled as axially rigid because a rigid floor diaphragm was assumed and the beam forces were not of interest for this study. The same damping model was used as for the CRSBF.

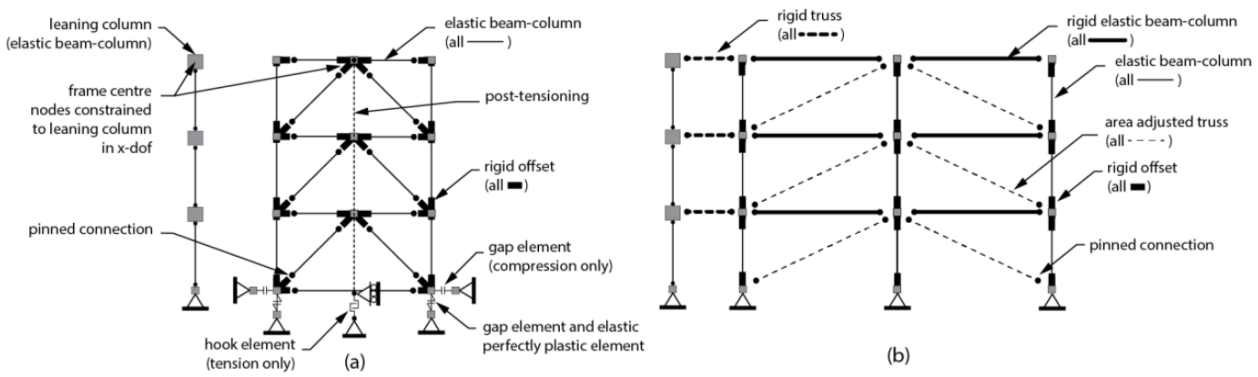

Fig. 3 – Schematic of frame models: (a) CRSBF [after 17], (b) BRBF

4. Ground Motion Selection and Scaling

The suite of 44 far-field ground motions from FEMA P695 [31] was used for this study. It was scaled to minimize the mean squared error of the median spectral acceleration compared to the DE between 0.2 times the shortest first-mode period of any of the designs and twice the longest period, so as to allow demands to compared for different frames with the same ground motions. The resulting 5% damped pseudo-acceleration spectra are shown in Fig. 4.

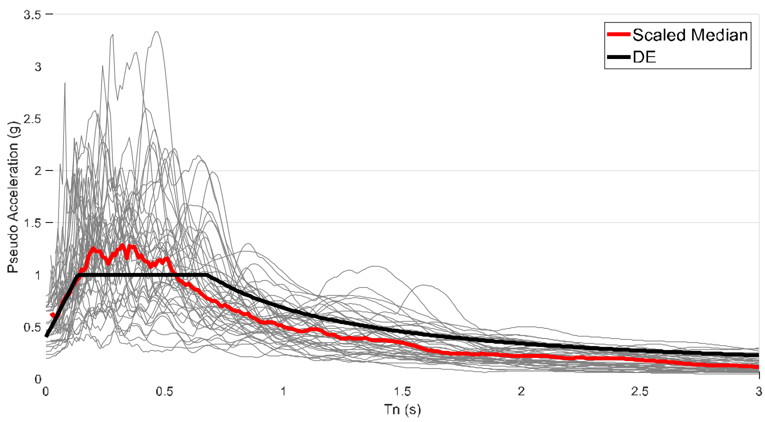

Fig. 4 – Target and scaled 5% damped ground motion pseudo-acceleration spectra

5. Comparison of Structural Behaviour

5.1 Peak Interstorey Drifts

The top row of Fig. 5 shows the peak interstorey drifts in all three frames during the suite of ground motions scaled to the design level. As described in Section 3, the designs were all tuned so that the largest median peak interstorey drift would be 1.5%. Even so, significant differences are seen between the different systems. Both CRSBFs are dominated by the rocking mode, with a nearly uniform peak interstorey drift profile during every ground motion, whereas the BRBF tends to have larger drift demands at the lowest level. While the uniform drift profile of the CRSBF makes it easier for a designer to reduce the peak interstorey drifts if desired, the non-uniform drift profile of the BRBF means that it may result in less damage overall to displacement-sensitive non-structural components, since these components would experience lower demands at some levels relative to those in the CRSBF. Considering the two CRSBF designs, while the median response is similar, the maximum values across all ground motions are somewhat larger for the design with less energy dissipation.

5.2 Residual Interstorey Drifts

The middle row of Fig. 5 shows the residual interstorey drifts for all three frames after the same ground motions. As intended by design, the residual drifts of both CRSBFs are zero. While the median residual drifts of the BRBF are less than 0.5% at all levels, suggesting that it may be more economically desirable to repair the BRBFs than to replace them at this excitation level [9], several ground motions lead to residual interstorey drifts of 1% or more at the bottom level, and even for lower residual drifts, structural repairs are expected to be more expensive, invasive, and time-consuming for the building with BRBFs than for the CRSBF designs. This could lead to an increase in overall building repair costs and downtime with the BRBF.

5.3 Demands on Brace Elements

The bottom row of Fig. 5 shows that, for all ground motions, the peak forces in each brace for the CRSBF designs are less than their capacities. This confirms that the design process was effective in avoiding structural damage in the CRSBFs, and that the modelling approach that used linear elastic elements for CRSBF members is valid. The demands and capacities are largest at the top level because the post-tensioning is anchored at that location.

The bottom right plot in Fig. 5 shows the hysteretic response of a BRB at the bottom level during the Cape Mendocino (Rio Dell Overpass Station) ground motion, which produced representative demands. At the DE intensity, the BRB yielded to a peak strain of 0.8%, enough to make the behaviour significantly nonlinear but not in the range where the model would become invalid due to possible BRB fracture. At one quarter of the DE intensity, there was no significant yielding.

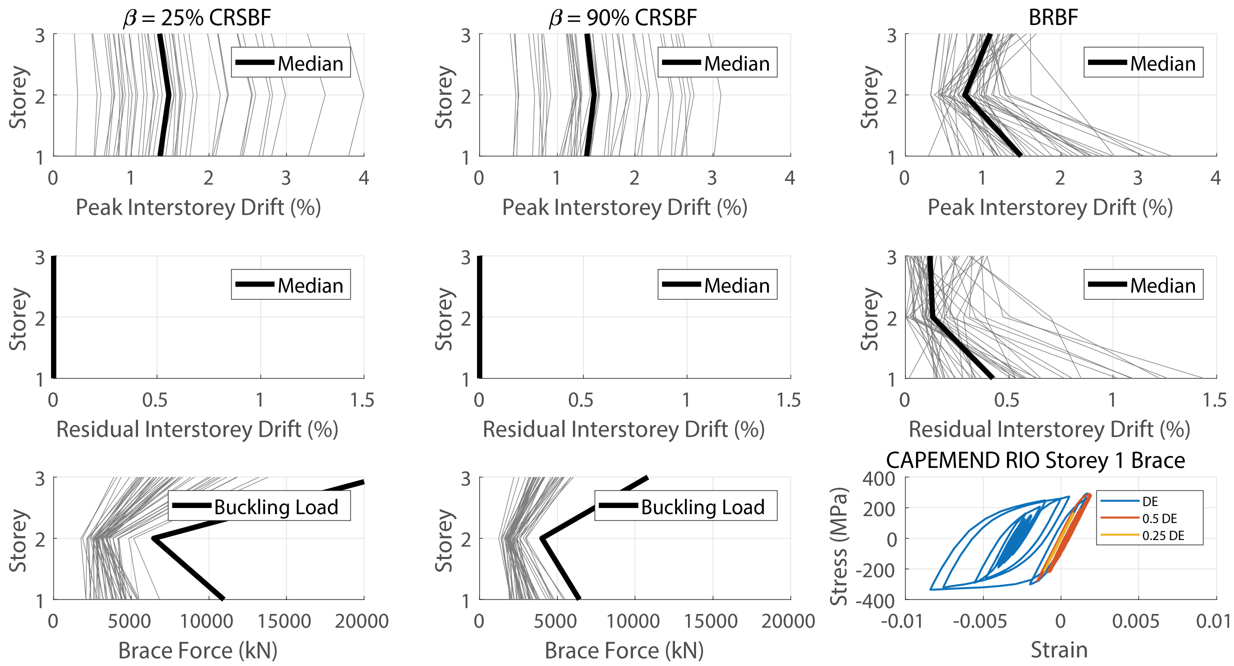

Fig. 5 – Structural response of frames: (top row) peak interstorey drifts; (middle row) residual interstorey drifts; (bottom row) peak braces forces in CRSBFs and hysteretic response of first-storey brace in BRBF

6. Floor Response Spectra

6.1 Comparison of Demands in CRSBF and BRBF

Figure 6 shows the median 2% damped floor pseudo-acceleration spectra that were computed for the three designs under the suite of ground motions, which provide an indication of the expected demands on acceleration-sensitive attached non-structural components such as piping systems and some mechanical equipment. The floor pseudo-acceleration spectra with both CRSBF designs were characterized by very large peaks, which occurred at periods somewhat longer than the second- and third-mode periods of the fixed-base CRSBF. These periods were found to correspond well with the higher-mode periods of the CRSBF modelled with boundary conditions to simulate rocking, which are given in Table 1. These peaks occur between 0.1 s and 0.3 s for these designs, which is likely to be relevant for some relatively flexible attached non-structural components such as mechanical equipment and electrical equipment cabinets [22], and have values that exceed 6 g at some floor levels.

Conversely, the floor pseudo-acceleration spectra with the BRBF design are characterized by a capping and spreading effect due to the elongation of periods as the BRB cores yielded. As such, while some peaks were observed near the modal periods of the BRBF, these peaks were less than 3 g at all levels. Considering periods above the second-mode periods of the CRSBFs, there was a crossover point where the pseudo-accelerations became similar or larger in the BRBF design compared to the CRSBF designs. Although not shown here, at periods greater than 1 s, the pseudo-accelerations once again became higher in the $\beta = 25\%$ CRSBF than the BRBF, with the largest difference being about 1.25 g occurring at a natural period of 1.25 s. This range of periods characterizes some flexible, hanging, or vibration isolated non-structural components.

When considering a reduced intensity of one quarter of the DE, the reduction in floor spectra differed between the CRSBFs and the BRBF. The floor spectra peaks in the buildings with CRSBFs were greatly reduced at the lower intensity: on average, the peaks near modes two and three were reduced by 72% and 44%, respectively. The reduction was less significant in the BRBF, with reductions of 20% and 38% at modes two and three, respectively. In fact, the distribution of the BRBF floor spectra at ¼ DE began to match the trends of the CRSBF, with more defined peaks in spectra at the periods of the structure. At the fundamental mode of the three-storey BRBF, the median floor spectra on average were actually 0.26 g larger at the ¼ DE intensity level than at DE. This is because, at the lower earthquake intensity, the BRBF experienced minimal yielding of the BRB cores and was not relieved from vibrating in its elastic modes for the majority of ground motions (similar to the CRSBFs), as shown in Section 5.3. Nevertheless, the CRSBFs reached their floor spectral peaks at shorter periods than the more flexible BRBF (see Table 1), and these shorter periods are expected to be relevant to relatively more acceleration-sensitive attached non-structural components [22].

6.2 Influence of CRSBF Energy Dissipation and Stiffness

Considering the two CRSBF designs, Fig. 6 shows that the floor spectra peaks are somewhat larger at the roof and second-floor levels for the design with more energy dissipation, while the peaks were larger at the third floor in the design with less energy dissipation. This is counter-intuitive, as more supplemental energy dissipation is often thought to reduce all aspects of the seismic response. However, the two CRSBF designs also differed in their rocking moment (related to R in Table 1) and stiffness (related to T_1 in Table 1). Thus, to better distinguish between the influences of these parameters, the analysis of the $\beta = 90\%$ CRSBF was repeated using the stiffer frame from the $\beta = 25\%$ CRSBF design, so as to compare the response with different amounts of post-tensioning and supplemental energy dissipation but identical frames.

For this case, Fig. 7 shows that the floor spectra are very similar when the frames are the same. The peaks near the second-mode period are within 10% of each other at all levels when the frames are identical, although the peaks near the third-mode period are significantly lower for the design with more supplemental energy dissipation. This suggests that the stiffness of the frame, which responds elastically, is the key factor in determining the floor spectra, while the parameters that influence the hysteretic response of the system (refer to Fig. 1) are relatively less influential.

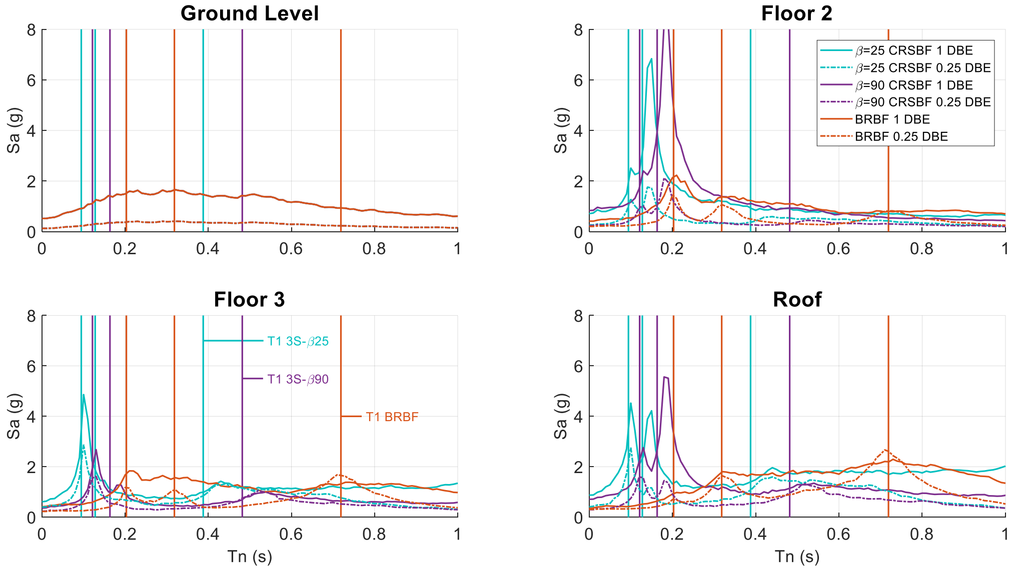

Fig. 6 – Median 2% damped pseudo-acceleration floor spectra

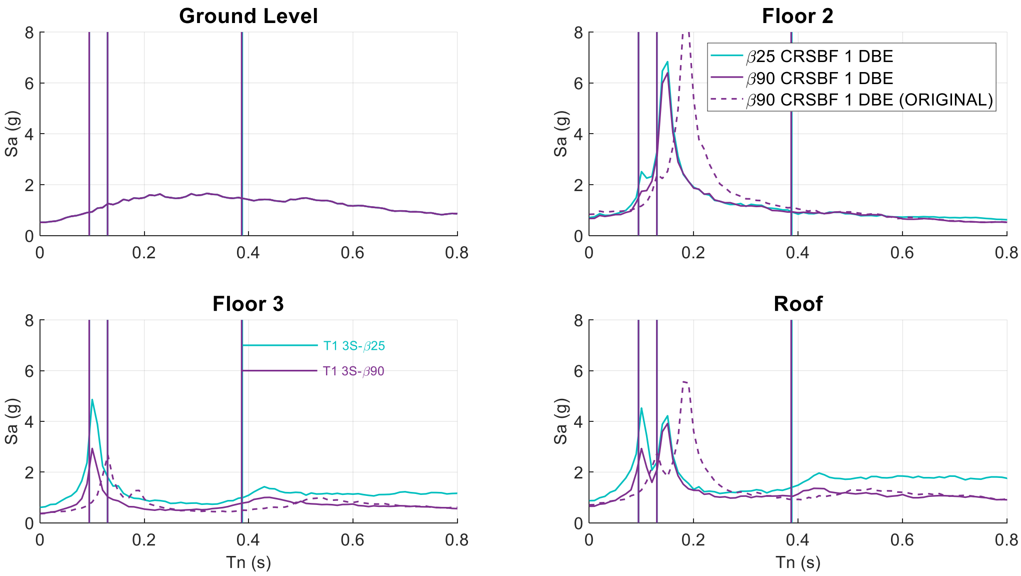

Fig. 7 – Median 2% damped pseudo-acceleration floor spectra for CRSBFs with identical frame members

7. Response after Impact on Foundation

As noted in Section 2, a key question related to the demands on non-structural components in CRSBFs is whether the abrupt stiffness increase when the frame impacts the foundation causes unusually large acceleration demands. To address this question, Fig. 8 shows the time history of several key response parameters for the $\beta = 25\%$ CRSBF design during a representative ground motion. These parameters are the column uplift, accelerations at all floor levels, and accelerations of attached non-structural components with periods of 0.44 s and 0.15 s, which are where the floor spectra peaked nearest the periods of the first two modes in Fig. 6. Vertical lines indicate instants of column impact, when the stiffness increases abruptly. This figure shows no indication of increased demands after instants when the CRSBF columns impact the foundation. Instead, the floor accelerations have a consistent vibration that appears to have an underlying shape that is similar to that of the column uplift time history, with significant oscillations with a period of approximately the second-mode period of the frame (T_2 = 0.13 s, $T_{2,rock}$ = 0.14 s). The attached non-structural components both vibrate primarily at their own natural periods, with no apparent increases in demand after column impact. These observations confirm and extend what was observed experimentally by Wiebe et al. [4].

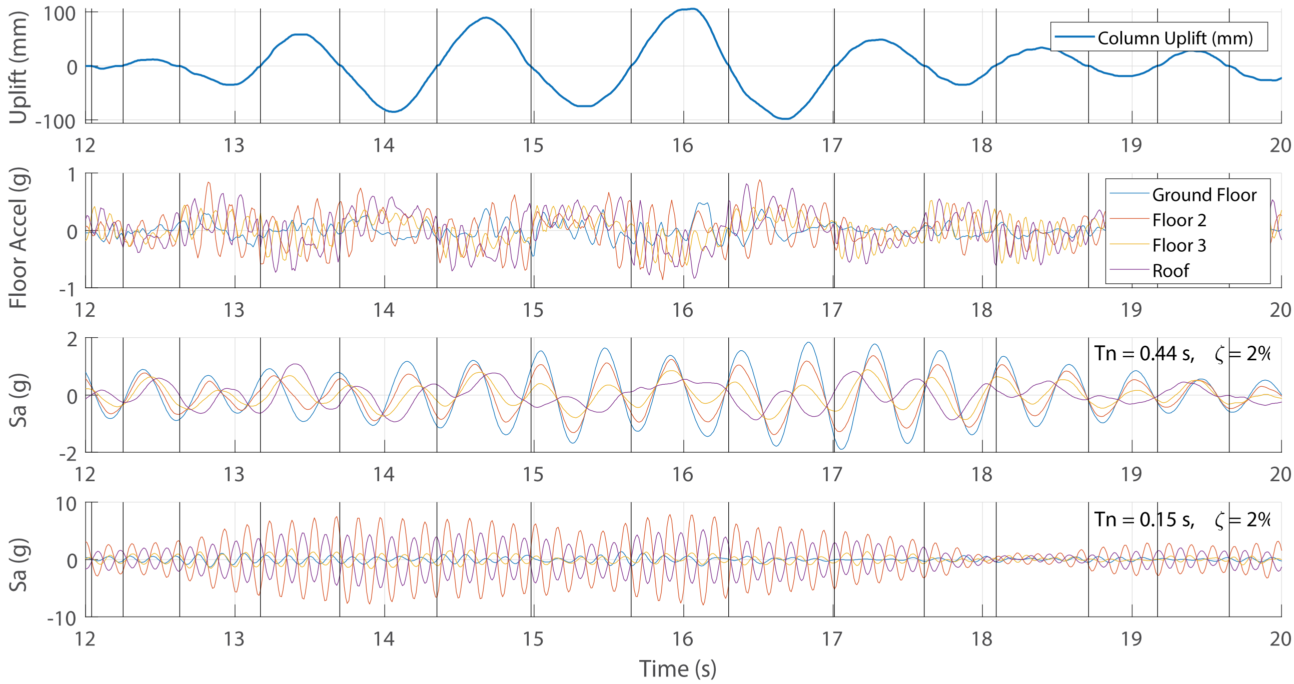

Fig. 8 – Response of $\beta = 25\%$ CRSBF design during representative ground motion: column uplift, floor accelerations, and pseudo-accelerations of attached non-structural components. Vertical lines indicate instants of column impact on foundation.

8. Conclusions

This paper added to the available literature on floor response spectra in buildings with CRSBFs by presenting results for a three-storey building located in a region of high seismicity. Two alternative designs were considered with different amounts of supplemental energy dissipation, leading to different frame designs, and one reference buckling restrained braced frame. All systems were tuned to achieve similar peak interstorey drifts of 1.5%, although this limit was nearly reached at all levels of the CRSBF but only the bottom level of the BRBF. As intended, the CRSBFs had essentially zero residual drifts, whereas the BRBF had residual drifts that ranged from essentially zero to nearly 1.5%, depending on the ground motion and storey level. As such, the demands on drift-sensitive non-structural components would be expected to be similar or less in the BRBF relative to the CRSBFs, but structural repair costs and downtime for the BRBF would add to the total damage relative to the CRSBF with its more constrained structural damage.

Attached non-structural components with short periods were observed to have much larger spectral pseudo-acceleration demands in the buildings with CRSBFs than in the building with BRBFs. The floor pseudo-acceleration spectra computed for the buildings with CRSBFs showed clear peaks at periods of less than 0.2 s, which is expected to be relevant for many acceleration-sensitive attached non-structural components. These peaks occurred near the higher-mode rocking periods, and compared to the spectral peaks in the building with BRBFs, they were much higher but over a smaller range of periods. These peaks appeared to be instigated by the elastic vibration of the CRSBFs in their higher modes, which was modified but not prevented by the rocking at the base. In contrast, the peaks were not greatly influenced by the amount of supplemental energy dissipation provided to the CRSBFs, nor could any evidence be identified to suggest that these spectral pseudo-acceleration demands were instigated by the abrupt stiffness change when the frame columns impact the foundation. As such, it appears inherent to the CRSBF system that the structural benefit of ensuring elastic response of frame members has the unintended consequence of increased demands on certain acceleration-sensitive attached non-structural components. This observation is limited to CRSBFs with rocking only at the base, whereas it could be expected that using multiple mechanisms to reduce the force demands on structural elements would also reduce the demands on acceleration-sensitive attached non-structural components. Further study is needed to explore this possibility.

9. Acknowledgements

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