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# SEISMIC COLUMN SPLICE DEMANDS IN BRACED FRAMES WITH BUCKLING CONTROLLED BRACES

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Numerous experimental tests showed that ductile square hollow structural sections (HSS) are susceptible to premature fracture under design-level earthquake ground motions. In addition, recent research has highlighted the vulnerability of column splices to reach their full strength once ductile square HSS braces experience a fracture in braced frames. In this study, the performance of column splices in 5- and 13-story special concentrically braced frames (SCBFs), with X-bracing configuration, located at a distance permitted by the seismic design provisions (i.e., four feet over the concrete floor level), is evaluated under a large number of ground motion excitations. The performance of identical SCBFs with conventional HSS braces and innovative buckling controlled HSS braces is compared. Overall, results from this research indicate that the prevention of brace buckling in SCBFs yields a substantial reduction in the seismically induced bending demands on the column splices, leading to a large margin of safety to remain elastic.

*Keywords*: Splice connections, Conventional braces, Innovative braces, Seismic demands.

## 1 INTRODUCTION

The failure of column splices endangers the integrity of gravity and lateral systems, compromising the overall structural stability. Column splices in SCBF systems are expected to endure seismic demands without undergoing inelastic deformations during a seismic hazard event (AISC 341 2016). In the design procedure of SCBFs, the required strength of column splices depends on the sizes of the connected columns that are designed using the expected load-carrying capacity of the brace members. That is to say, the design of a column splice in an SCBF is based on the capacity of the connected column determined from plastic analyses specified in AISC 341 (2016), considering inelastic tensile and compressive capacities of ductile braces.

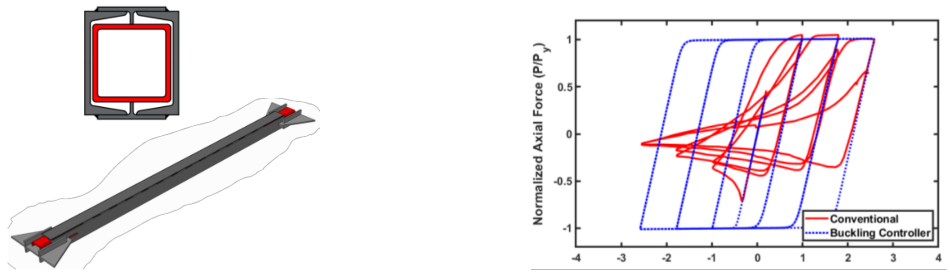
It is also evident from the recent studies conducted by Faytarouni *et al.* (2020a, 2020b) that the demand on column splices is highly dependable on the brace performance. Although braces are expected in seismic provisions to withstand large ductility demands without experiencing fracture at design level earthquake ground motions, experimental results (Faytarouni *et al.* 2019) have shown that braces, distinctly cold-formed HSS, are vulnerable to premature fracture under expected demand levels. Therefore, it was essential to assess SCBFs seismic requirements for column splices, ensuring whether the expected margin of safety to remain elastic under different demand levels is provided. On this account, an initiative was taken by Faytarouni *et al.* (2020a, 2020b) to evaluate the induced demands on SCBF column splices. The overarching conclusion was that column splices designed following the current seismic design requirements appeared to undergo demands larger than expected with the possibility of reaching full capacity, especially when braces experience fracture.

For existing or new SCBFs, a potential way to mitigate column splice demands could be through enhancing the fracture life of braces since premature fracture of braces tends to impose additional splice demands (Faytarouni *et al.* 2020b). To examine this, a novel cost-effective buckling controller device proposed by Seker *et al.* (2019) to elongate the fracture life of cold-formed square HSS members is incorporated in conventional SCBF. Note that the developed buckling controller, called channel-encased braces, showed promising results in mitigating the drift and brace ductility demands, leading to an improved overall structural performance of existing SCBFs. The objective of this paper is to study the effect of SCBFs equipped with buckling controllers on column splice demands.

## 2 BUCKLING CONTROLLER

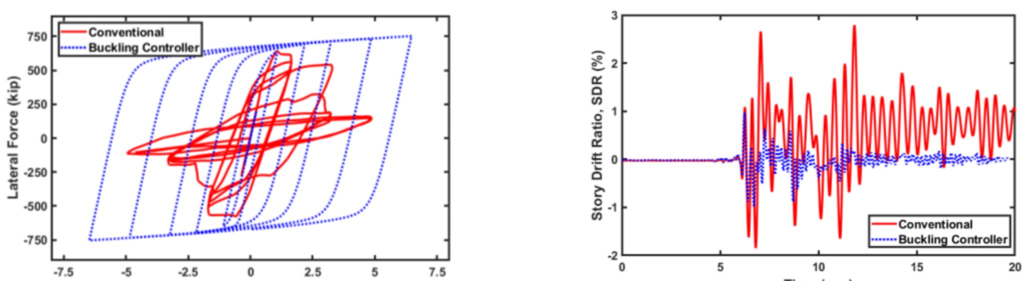
To ensure that braces undergo large post-buckling axial deformations without premature fracture under severe earthquake excitations, seismic provisions require proper detailing for ductility that is mainly attained by limiting the width-to-thickness and slenderness ratios to the code-specified limits. Despite meeting member ductile requirements, braces, particularly those made with square HSS, have shown experimentally to be prone to early fracture, leading to significantly lower energy dissipation capability of CBFs, eventually resulting in excessive demands on beams and columns. In an attempt to improve the energy dissipation capacity of square HSS members in existing CBFs, Seker *et al.* (2019) proposed a buckling controlling device capable of mitigating the likelihood of local-buckling-induced fracture until a desired inelastic response level is achieved. Figure 1(a) shows the buckling controller that is composed of two channels attached through a fillet welded tee-shapes, encasing square HSS brace. The buckling controller's effectiveness is demonstrated herein when applied to square HSS from component to system-level performance through quasi-static and dynamic loading. On a component level, the specimen tested by Fell *et al.* (2009), namely HSS1-1, was employed to compare the hysteretic response of conventional square HSS and channel-encased braces. Figure 1(b) plots the response of HSS1-1 in red and the same specimen's response but including the buckling controller in blue. As indicated in Figure 1(b), the buckling controller seemed to successfully control both global and local buckling, leading to increased energy dissipation of existing square HSS. The buckling controller improved the cyclic stability of the brace significantly.

On a system level, braced frames tested quasi-statically by Uriz (2008), shown in Figure 2(a), and dynamically by Okazaki *et al.* (2013), shown in Figure 2(b), were utilized to compare the influence of buckling controller on the system behavior. Figure 2(a) illustrates the overall frame response of Uriz's frame, plotted with respect to base shear versus upper beam displacement with and without buckling controller. After introducing the buckling controller, the frame's lateral strength and stiffness enhanced with no signs of deterioration, not to mention the substantial improvement in energy dissipation capacity. Figure 2(b) shows the story drift ratio response of Okazaki's tested frame with and without buckling controller. The reduction of drift demands is apparent in Figure 2(b) when using the buckling controller. With conventional braces, the frame underwent maximum drift demands of about 3% and around 1% of residual drift. However, applying the buckling controller, the frame's maximum drift was almost 1%, with a stable elastic response throughout the history. Therefore, it is evident that the developed buckling controller can mitigate the possibility of experiencing less ductile failure modes associated with conventional square HSS braces.




(a) (b)

Figure 1. Isolated brace response: (a) brace with the buckling controller, and (b) hysteretic behavior with and without buckling controller.

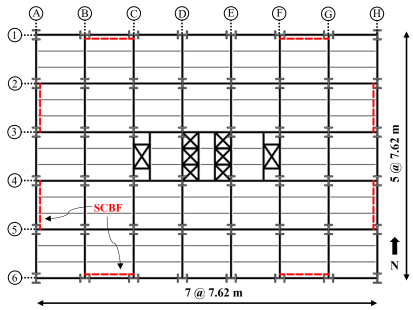



(a) (b)

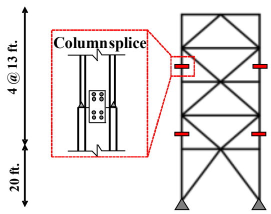
Figure 2. System level performance with and without buckling controller: (a) tested frame of Uriz (2008), and (b) tested frame of Okazaki *et al.* (2013).

## 3 CASE STUDY SCBF BUILDINGS

Two-office braced steel buildings, 5- and 13-story, were considered to investigate demands endured by column splices. For each building, two cases were investigated: the first case consisted of conventional braces, while in the second case, the braces were equipped with buckling controllers. The considered structures are shown in Figure 3. Each bay was 25 feet in length. Figures 3(a) and (d) draw the floor plan dimensions, where seismic force resistance was provided by four SCBFs for 5-story, and eight SCBFs for 13-story, in each orthogonal direction The first story's height was 20 feet and 13 feet for all remaining stories. Column splices in both structures were located at a distance of four feet over the finished concrete level, as recommended by the seismic provisions and shown in Figures 3(b) and (e). First, the SCBFs were designed with conventional square HSS braces following requirements specified in ASCE (2016) and AISC 341 (2016). Then, square tubing was encased in two channels attached with fillet welded stitches made with WT sections. The seismic design yielded the member sizes summarized in Figures 3(c) and (f). The frames with and without buckling controllers were analytically modeled in OpenSees (McKenna *et al*. 2000) and subjected to 20 earthquake ground motions. Readers are referred to Seker *et al.* (2019) for detailed information about the buckling controller design, analytical models, and selection of ground motion records.



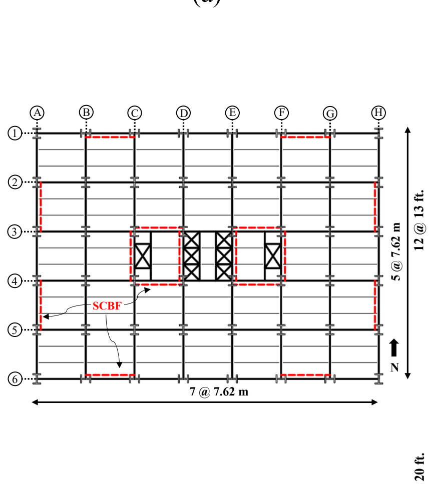

(a)



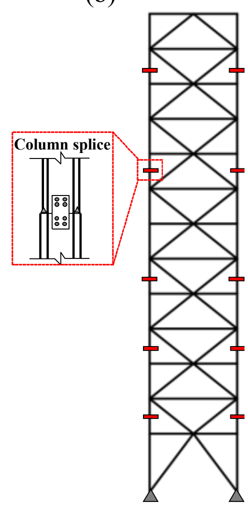

(b)

| Level | Braces | Columns | Beams | Channels | WT |
|---|---|---|---|---|---|
| 5 | HSS 5×5×1/2 | W 14×132 | W 21×248 | 2C 7x14.75 | WT 5x7.5 |
| 4 | HSS 7×7×5/8 | W 14×132 | W 18×65 | 2C 9x20 | WT 7x17 |
| 3 | HSS 7×7×5/8 | W 14×257 | W 18×35 | 2C 9x20 | WT 7x17 |
| 2 | HSS 10×10×3/4 | W 14×257 | W 18×86 | 2C 12x30 | WT 7x30.5 |
| 1 | HSS 10×10×3/4 | W 14×283 | W 21×201 | 2C 12x30 | WT 7x30.5 |

(c)




(d)




(e)

| Level | Braces | Columns | Beams | Channels | WT |
|---|---|---|---|---|---|
| 13 | HSS 4×4×1/2 | W 14×132 | W 21×182 | 2C 5x9 | 2WT 4x6.5 |
| 12 | HSS 6×6×1/2 | W 14×132 | W 18×60 | 2C 8x18.75 | 2WT 4x12 |
| 11 | HSS 6×6×1/2 | W 14×233 | W 18×35 | 2C 8x18.75 | 2WT 4x12 |
| 10 | HSS 7×7×5/8 | W 14×233 | W 18×71 | 2C 9x20 | WT 7x17 |
| 9 | HSS 7×7×5/8 | W 14×233 | W 18x35 | 2C 9x20 | WT 7x17 |
| 8 | HSS 8×8×5/8 | W 14×398 | W 18×71 | 2C 10x25 | 2WT 5x19.5 |
| 7 | HSS 8×8×5/8 | W 14×398 | W 18×35 | 2C 10x25 | 2WT 5x19.5 |
| 6 | HSS 8×8×5/8 | W 14×398 | W 18x86 | 2C 10x25 | 2WT 5x19.5 |
| 5 | HSS 8×8×5/8 | W 14×550 | W 18x35 | 2C 10x25 | 2WT 5x19.5 |
| 4 | HSS 9×9×5/8 | W 14×550 | W 18×86 | 2C 12x30 | WT 7x30.5 |
| 3 | HSS 9×9×5/8 | W 14×665 | W 18×35 | 2C 12x30 | WT 7x30.5 |
| 2 | HSS 10×10×3/4 | W 14×665 | W 18×86 | 2C 12x30 | WT 7x30.5 |
| 1 | HSS 10×10×3/4 | W 14×730 | W 21×201 | 2C 12x30 | WT 7x30.5 |

(f)

Figure 3. Considered building frames: (a) 5-story plan view, (b) 5-story column splices, (c) 5-story section dimensions, (d) 13-story plan view, (e) 13-story column splices, and (f) 13-story section dimensions.

## 4 COLUMN SPLICE DEMANDS WITH AND WITHOUT BUCKLING CONTROLLERS

Seismically induced demands on splices in the considered cases of SCBFs (i.e., conventional braces and braces equipped with buckling controllers) were assessed by means of tensile axial forces normalized by yielding capacity ($P_t/P_y$) and flexural forces normalized by moment capacity ($M/M_p$). Figures 4(a) and (c) exemplify the peak axial force demand at splice $P_t/P_y$ for both SCBFs with and without buckling controller under GM3 (1983 Coalinga record). Similarly, the peak bending moment demands ($M/M_p$) are plotted in Figures 4(b) and (d) at the splices. The dark and light fill in Figures 4(a) and (c) represent axial forces normalized by yielding and buckling. In Figures 4(b) and (d), the moment was plotted on the compression side. By examining Figure 4, one can observe that with conventional braces, the peak $P_t/P_y$ occurred at splice 1 for the 5-story and at splice 3 for the 13-story with axial demands reaching 40% and 37% of yielding capacity, respectively. In contrast, both structures' splices exhibited higher axial demands, achieving 56% and 58% yielding capacity when buckling controllers were introduced. Unlike the axial force demands, column splices, when buckling controllers were employed, experienced lower flexural demands. As indicated in Figures 4(b) and (d), the buckling controllers appear to substantially reduce the $M/M_p$ ratios by almost half compared to the case with conventional braces.

The peak $P_t/P_y$ and $M/M_p$ responses attained by any splice in the 5- and 13-story SCBFs with and without buckling controllers under the applied ground motion records are presented in Figure 5. Referring to the summary of $P_t/P_y$ response given in Figures 5(a) and (c), the peak axial demand ratios, $P_t/P_y$, in the 5- and 13-story SCBFs with buckling controllers were consistently higher than those in SCBFs with conventional braces. The $P_t/P_y$ response without buckling controllers appears to be the same, with an average of 0.38 in the 5- and 13-story SCBFs. With buckling controllers, splices experienced greater tensile axial demands with an average $P_t/P_y$ of 0.55 for both frames. This increase in the axial demands might be mainly due to the increased lateral strength of the frames with buckling controllers, as exemplified in Figure 2(c). On the other hand, the splices in 5- and 13-story SCBFs with buckling controllers exhibited lower flexural demands over all the applied excitations. The average ratio of $M/M_p$ without buckling controllers was 0.44 and 0.57 for the 5- and 13-story, and with buckling controllers, the average reduced to nearly half, 0.2, and 0.25. This could be attributed to the reduction in the story drift response of both buckling controlled frames, reflecting on system response's influence on the flexural demands endured by splice connections.

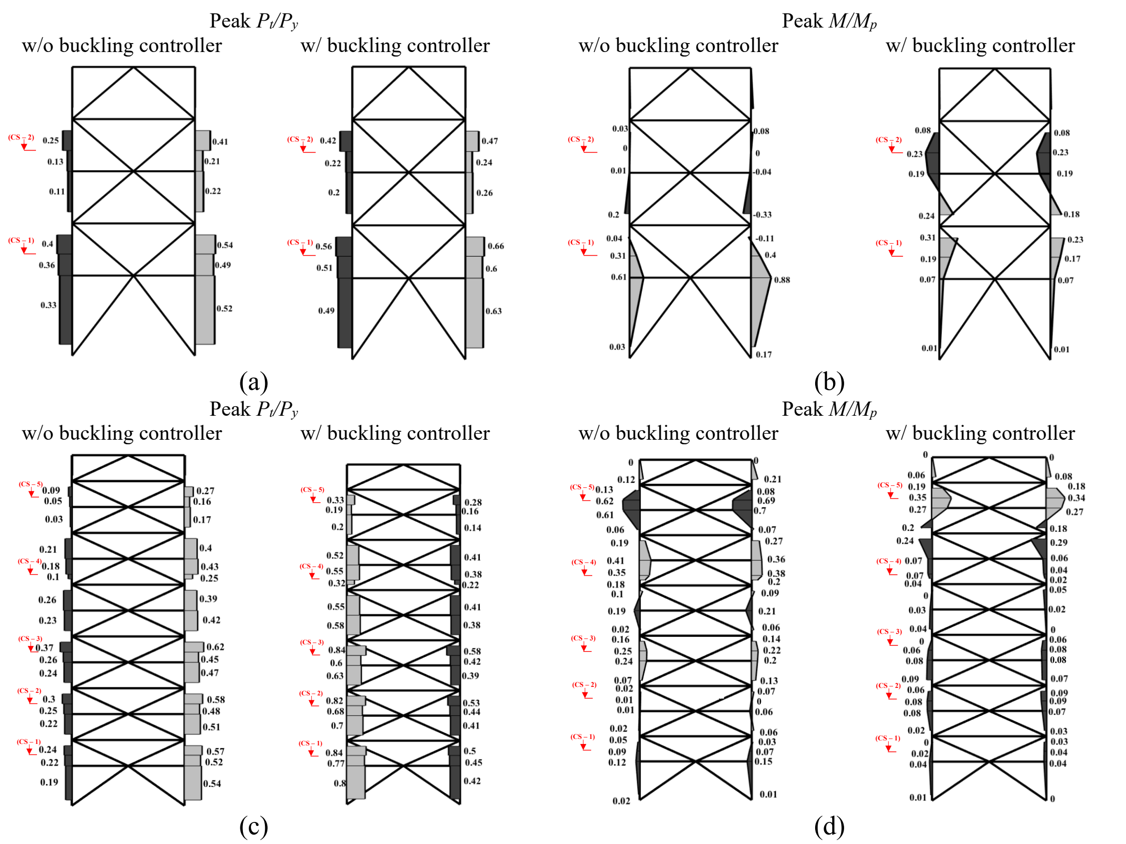



Figure 4. Normalized demands on columns with and without buckling controller under GM3: (a) 5-story at peak $P_t/P_y$, (b) 5-story at peak $M/M_p$, (c) 13-story at peak $P_t/P_y$, and (d) 13-story at peak $M/M_p$.

## 5 CONCLUSIONS

The splices behaved consistently with the design intention when buckling controllers are utilized. After introducing the buckling controllers, the axial forces endured by splice connections remained with a convenient margin of safety despite the increase in tensile demands, and most importantly, flexural demands were mitigated to extents lower than the minimum required flexural strength stipulated in the provisions, which is 50% the plastic moment capacity of the smaller connected column. Further analyses are needed to generalize the conclusions drawn in the present paper.

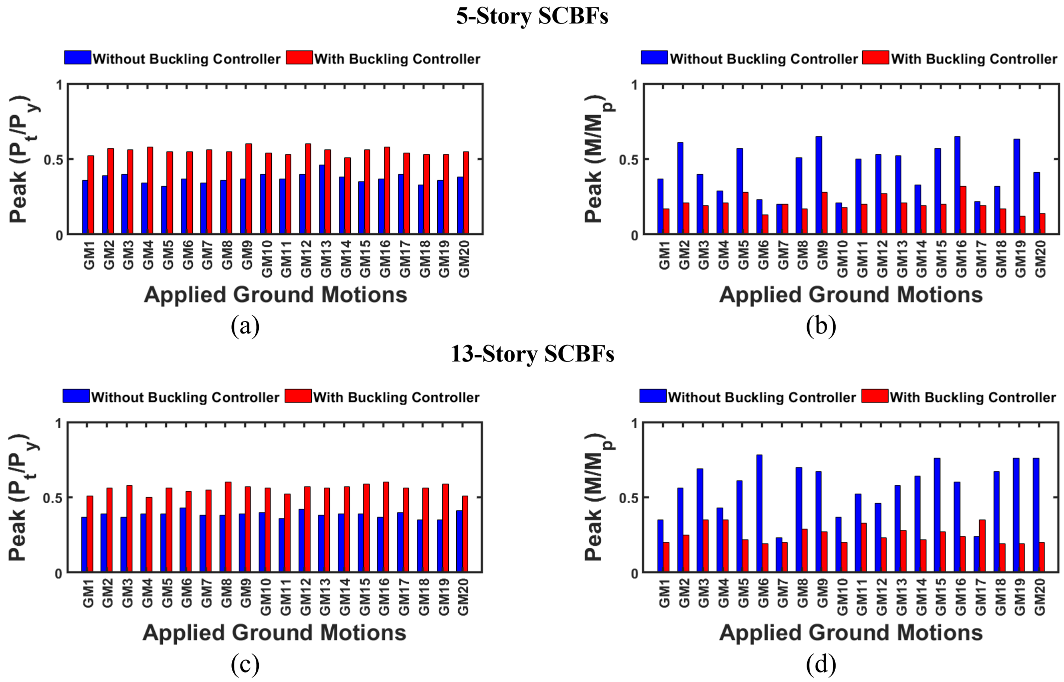



Figure 5. Column splice response with and without buckling controller under applied earthquakes: (a) 5-story at peak $P_t/P_y$, (b) 5-story at peak $M/M_p$, (c) 13-story at peak $P_t/P_y$, and (d) 13-story at peak $M/M_p$.

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