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BUCKLING RESTRAINED BRACE CONNECTION AND STABILITY PERFORMANCE ISSUES

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Abstract

Buckling restrained braces (BRB) are a popular seismic lateral force resisting system. While some tests indicate excellent performance of BRBs, studies in recent years have demonstrated premature failure mechanisms which can occur at loads/displacements significantly less than those anticipated by conventional design checks. This paper lists published BRB and BRB system failures that have occurred around the world during testing. These have included sway buckling of the gusset, plastic hinging at the restrainer end, gusset weld fractures, local buckling and fracture of the adjacent framing members, and restrainer wall bulging. Simple theory is used to explain why each of the failures occurred. This information is used to better inform the design process.

As a result of the study, it is shown that some commonly used design procedures are not appropriate. As an example, the use of an effective length factor $k_e < 1$ for the gusset plate that can fail by sway may be non-conservative. Also, state-of-the-art design methods to obtain reliable BRB system performance under expected deformations are provided.

Keywords: buckling-restrained brace system, out-of-plane stability, gusset weld fracture, restrainer bulging

1. Introduction

Buckling-restrained braces (BRB) are widely used in the United States, Japan and other seismic countries and have become increasingly popular in the Christchurch, New Zealand rebuild, often replacing more traditional structural systems such as moment frames, concentrically braced frames and eccentrically braced frames. This is because they are perceived to have "low damage" characteristics allowing them to behave well during not only the main earthquake, but also during significant aftershocks as well. When combined with a secondary lateral system to provide post-yield stiffness and limit residual drifts, rapid re-occupancy can be achieved.

Design responsibility is commonly split, with engineering consultants designing the system and suppliers designing the BRB. Thus, guides and standards usually emphasize the system-level design [1-3], with several exceptions where some detailed provisions have also been provided for the BRB [4-6]. Regardless, in almost all jurisdictions the brace itself is treated as a proprietary product that must conform to testing requirements.

Product validation testing has resulted in a large number of successful uniaxial and subassembly tests. The majority of the subassembly tests have been conducted under in-plane loading. It is worthwhile to note that these may suffer from a reporting bias with only successful tests reported due to their proprietary nature. Nevertheless, several independent full-scale 3D frame [7-9] and numerous subassembly tests have demonstrated that well-designed BRBs can perform satisfactorily up to design drifts. Good performance was also noted in the 2011 Tohoku, Japan earthquake.

An example from the Tohoku earthquake is the 1990s-vintage tall building shown in Fig.1 [10], which is located in Koriyama, Fukushima Prefecture (near K-Net station FKS018). While this building is also fitted with viscoelastic dampers, the BRBs contributed to the performance and were subjected to moderate ductility demands of $\mu \approx 4$ during the main strong ground motion. Less non-structural damage was observed in this structure relative to those adjacent and none of the BRBs required replacement, with the remaining fatigue life validated using attached cumulative displacement meters.

The lateral system consists of a bidirectional moment frame with mortar filled steel tube BRBs and viscoelastic dampers distributed in the bottom and middle thirds, respectively. A typical bay is shown Fig.1, although chevron configurations were also used.

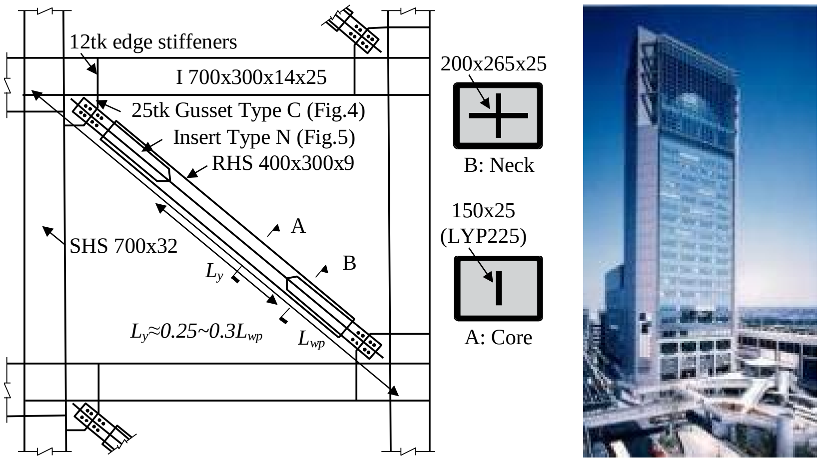


Fig. 1 – BRB from Tohoku 2011 [10]

Generally, BRBs are a reasonably well tested system that can achieve superior performance when the system is carefully designed and detailed by informed engineers and reputable suppliers. However, the unique characteristics of these braces can produce several undesirable failure mechanisms, which are directly influenced by decisions made for the adjacent framing, connections and restrainer. Therefore, a good system-level design requires the engineering consultant to also understand the nuanced details of the BRB.

Detailed studies in recent years have demonstrated particular mechanisms which can occur at loads and displacements significantly less than anticipated by conventional design checks. In general, BRB design must anticipate a range of strength and stability considerations, including both local and global effects as shown in Fig.2.

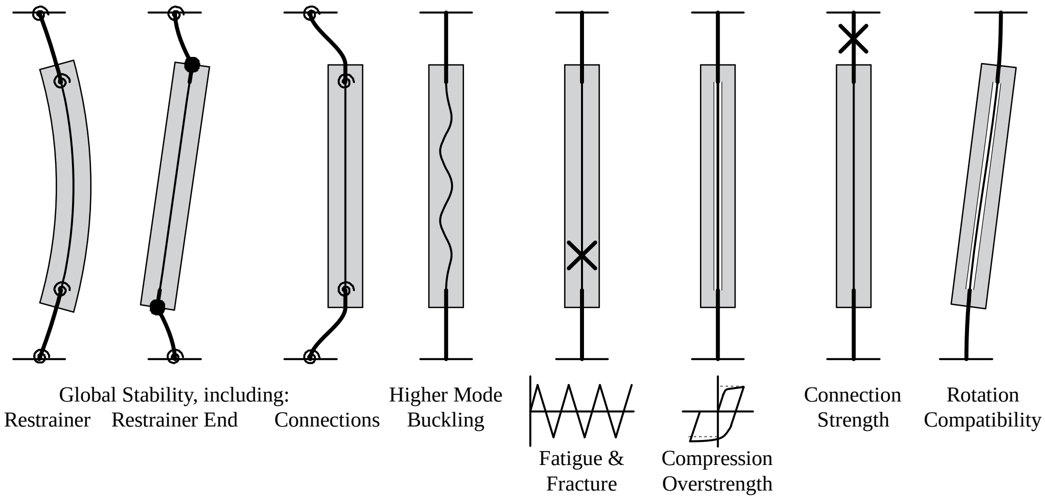


Fig. 2 – BRB Stability and Strength

This paper is primarily concerned with BRBs consisting of a steel yielding core restrained by a mortar filled steel tube and connected with gussets. Selected recent experiments are reviewed to demonstrate the key failure mechanisms for BRBs and BRB frames. Simple theory is used to explain why each of the failures occurred and state-of-the-art design and analysis procedures are reviewed. This information is used to better inform the design process.

Four design aspects will be reviewed in further detail:

1. *Global stability*, including the effects of connections
2. *Connection buckling* in the out-of-plane direction
3. *Connection strength*, including beam/column damage and connection weld fracture
4. *Higher mode buckling* leading to restrainer wall bulging

2. Global stability

2.1 Stability concepts

While the fundamental characteristic of BRBs is that buckling of the yielding core is suppressed by the debonded restrainer casing, sufficiently stiff connections are also required to supress global instability. Critically, recent research has demonstrated that the performance is sensitive to 1) gusset and connection zone flexural stiffness, 2) adjacent framing rotational stiffness and 3) restrainer end moment continuity [11]. For all three aspects the *out-of-plane* direction typically governs.

Stability design equations have become more advanced since the initial formulations, which were primarily focused on the restrainer requirements to suppress buckling over the yielding length and assumed stiff connections [12]. To classify instabilities observed during testing and to introduce a new system-level stability design method, the stability concepts (Fig.3) proposed in the AIJ [5] guidelines are described, which are differentiated by the assumed degree of moment transfer capacity provided at the restrainer ends:

Concept A: Hinges assumed at restrainer ends, with stability ensured through gusset cantilever action.

Concept B: Full moment continuity exists at restrainer ends, with overall or gusset sway buckling critical.

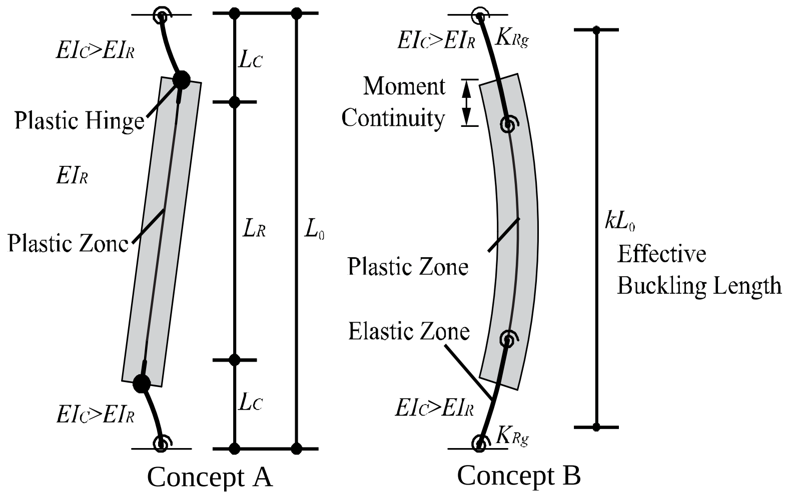


Fig. 3 – Stability Concepts modified from [5]

It is apparent that stability relies on interaction of the *whole* system. If the degree of moment continuity between the elastic neck portion of the brace and restrainer casing is low, then little rotational restraint exists at the connection tip (unlike concentric braced frames), and the behaviour primarily depends on the stiffness of the connection zone, gusset and adjacent framing (Concept A). If moment continuity is ensured by providing a large overlap at the restrainer end, the gusset's equivalent effective length is substantially reduced. Either gusset sway buckling or the overall buckling mode shown in Fig.3 may govern (Concept B).

As global stability is closely related to connection stability, current best practice is to consider the system as a whole. A companion paper reviews recent developments in stability assessment of BRBs and proposes a rigorous unified method developed in collaboration with major Japanese BRB suppliers [13].

2.2 Stiffness parameters

Gusset stiffener topology, adjacent framing configuration, and restrainer end embedment strongly influence out-of-plane stiffness, and are discussed below.

Gusset flexural stiffness affects both stability concepts and is dominated by out-of-plane stiffener layout. While BRB connection type and construction sequence may dictate which stiffener layouts are feasible, those with full-depth stiffeners (Type C & D in Fig.4) offer far more out-of-plane stiffness. Gusset types A (no additional stiffener) and B (partial depth stiffener) are common in the US and NZ, while Types C (full depth edge stiffeners) and D (full depth central stiffener) are more frequently used in Japan.

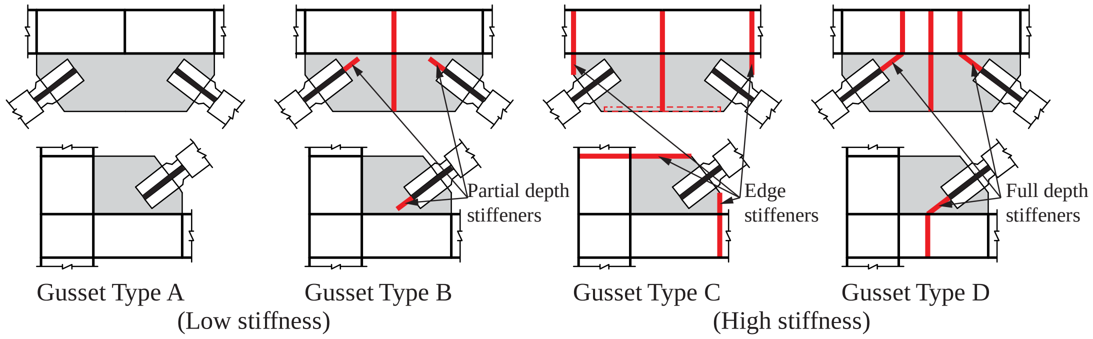


Fig. 4 – Gusset stiffener topologies

Adjacent framing provides out-of-plane rotational stiffness at the gusset base and is incorporated in the rotational stiffness term K_{Rg} from Fig.3. A bidirectional moment frame at beam/column connections and fixed end secondary beam at beam connections provides effectively rigid behaviour, though other configurations may introduce substantial flexibility:

- Chevron configuration beam connection adjacent to voids
- Chevron configuration beam connection with only a thin slab providing torsional restraint
- Perimeter single diagonal configuration with simply supported beams in the transverse direction

Restrainer end moment transfer capacity, and the potential for plastic hinges to form depends on the connection "neck" embedment or collar overlap length within the restrainer, as shown in Fig.5. A hinge forming in this zone has been observed to initiate buckling, a condition aggravated by the additional moments induced by out-of-plane deformations experienced during bi-directional seismic actions. A key parameter for the typical restrainer end condition of an embedded cruciform is the insert ratio of the neck embedment length L_{in} to neck out-of-plane depth D_n. L_{in} activates a longer transfer length, while D_n dominates the neck's out-of-plane moment capacity, and hence moment transfer demand. A significant embedment ($L_{in}/D_n \approx 2.0$) is typically needed to ensure continuity of the full connection moment capacity with the restrainer [11,13].

To maintain consistency with previous research, L_{in}/D_n is defined as the as-built insert ratio. For design, consideration should be given as to what tensile deformations are expected at initiation of buckling, which will reduce the effective insert length, L_{in}. Hinging mechanisms for both cruciform and collar restrainer end types are depicted in Fig.5. These are labelled by expected hinge mechanism for various degrees of moment transfer: Type P (Pin), R (Restrainer) and N (Neck). When full neck flexural capacity can develop in the (stronger) restrainer, hinging is supressed in the insert zone and the neck's moment $M_P^{r\text{-}neck}$ is achieved.

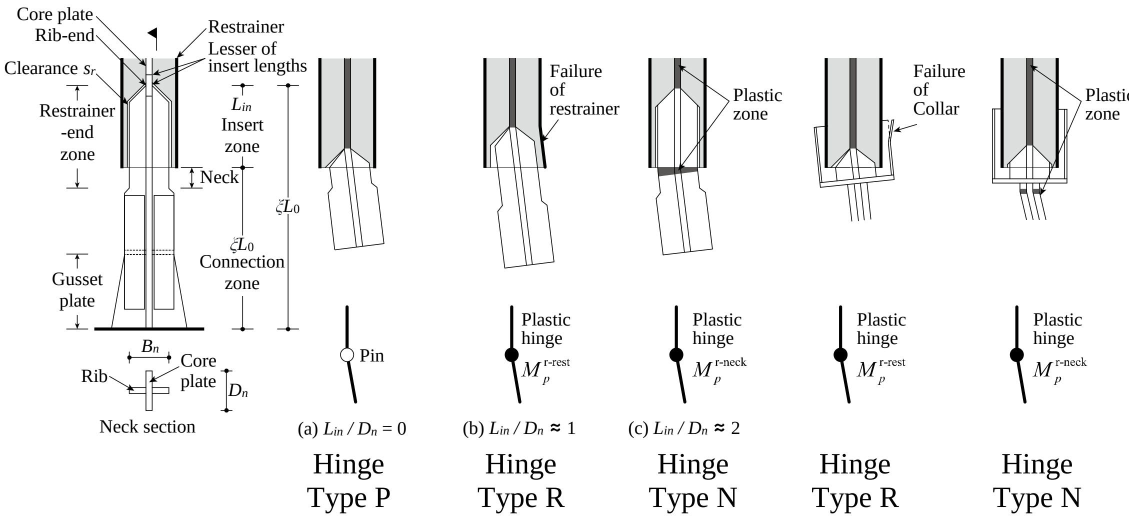


Fig. 5 – Restrainer end hinging mechanisms modified from [17]

Recent experiments have tended to observe instability manifested in a global buckling mode dominated by connection sway at one or both of the ends. These observations will be reviewed in the following section.

3. Connection out-of-plane buckling

3.1 Buckling with restrainer end hinging

When the restrainer end has insufficient moment transfer capacity, a hinge can form that leads to premature instability. This is typically characterized by rigid body or flexural out-of-plane buckling of the gusset. This mechanism has been observed in frame tests of both chevron and single diagonal configurations [14-18].

Chou *et al.* [14] tested 5 specimens in-plane in a bare frame with a single diagonal configuration, cruciform insert ratio of L_{in}/D_n<1.0 (Type P) and gusset Type C (Phase 1) and Type B (Phase 2). While the purpose of the test was to investigate gusset demands due to in-plane frame action, severe out-of-plane buckling with restrainer end hinging was observed at 0.7% drift during the Phase 2 test (gusset Type B).

Hikino *et al.* [15] tested 2 specimens in-plane in a bare frame with a chevron configuration, cruciform insert ratios of L_{in}/D_n=0.4 (Type P) and 1.3 (Type R), and gusset Type D. The purpose of this experiment was to study the effect of insert ratio on stability. The specimen with L_{in}/D_n=0.4 buckled, leading to system failure.

Takeuchi *et al.* [16] tested 6 specimens uniaxially with rigid boundary conditions in a single diagonal configuration, applying a 1% out-of-plane drift, with a specimen cruciform insert ratio of L_{in}/D_n=1.0 (Type R) or 2.0 (Type N), and gusset Type A or C. The effects of debonding gap (*s*=1~2mm), insert ratio, gusset stiffness and casing type (square, circular) are summarized in Table 1, along with the achieved compressive force P_{exp} and tensile yield force P_y:

Table 1 – Single diagonal buckling test results [16]

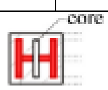
Specimen	L_{in}/D_n	Gusset	Debonding	Casing	Result	P_{exp} / P_y
MRL1.0S1H	1.0	Type C	1 mm	Rectangle	Stable	452 / 330kN
MRL2.0S1	2.0	Type A	1 mm	Rectangle	Stable	535 / 420kN
MRL2.0S2	2.0	Type A	2 mm	Rectangle	Buckled	507 / 420kN
MCL2.0S2	2.0	Type A	2 mm	Circular	Buckled	375 / 390kN
MRL1.0S1	1.0	Type A	1 mm	Rectangle	Buckled	362 / 420kN
MRL1.0S2	1.0	Type A	2 mm	Rectangle	Buckled	300 / 420kN

Takeuchi *et al.* [17] tested 6 specimens in a frame subassembly with chevron configuration, applying a 1% out-of-plane drift, with specimen cruciform insert ratio of L_{in}/D_n≈0 (Type P) or 2.0 (Type N), and gusset Types B or C. The effects of gusset stiffness, casing type (square, circular) and neck/insert strengthening measures are summarized in Table 2:

Table 2 – Chevron buckling test results [17]

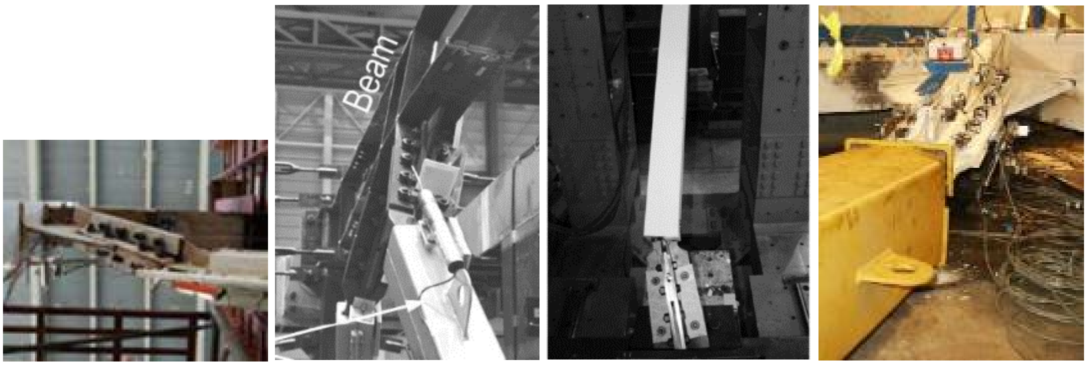
Specimen	L_{in}/D_n	Gusset	Strengthening	Casing	Result	P_{exp} / P_y
H-RN2	2.0	Type C	-	Rectangle	Stable	549 / 320kN
M-CN2	2.0	Type C	-	Circular	Stable	542 / 320kN
L-RN'2	2.0	Type B	-	Rectangle	Buckled	527 / 320kN
L-RF2	2.0	Type B	Rib Inserts[1]	Rectangle	Stable	564 / 320kN
L-CC2	N/A	Type B	Collar[2]	Circular	Stable	564 / 320kN
L-RN0	<1.0	Type B	-	Rectangle	Buckled	339 / 320kN

[1] Ribs form flanges of cruciform, with core parallel to gusset
[2] Collar 191ϕx5.3mm collar, with 180mm overlap



Palmer *et al.* [18] tested 6 specimens in-plane in a bare frame with a single diagonal configuration, cruciform insert ratio of L_{in}/D_n=0.9 (Type R) and gusset Type B. The purpose of the experiment was to study in-plane drift performance with varying gusset tapers and core orientations. In 5 specimens, local buckling and/or yielding of the adjacent framing was followed by out-of-plane instability with restrainer end hinging.

Fig.6 shows the buckling mechanisms for specific tests. Core tensile yield (P_y) and theoretical compressive ultimate (P_{uc}) force (including strain hardening and compression overstrength) are compared against the achieved compressive force (P_{exp}).



$P_y \approx 590$ kN	$P_y \approx 260$kN	$P_y \approx 290$kN	$P_y \approx 960$kN
$P_{uc} \approx 990$ kN	$P_{uc} \approx 450$kN	$P_{uc} \approx 430$kN	$P_{uc} \approx 1530$kN
$P_{exp} = 693$ kN	$P_{exp} = 309$kN	$P_{exp} = 300$kN	$P_{exp} = 1156$kN
(a) Specimen 2, Chou *et al.* [14]	(b) Specimen 2, Hikino *et al.* [15]	(c) MRL1.0S2, Takeuchi *et al.* [16]	(d) BRB01, Palmer *et al.* [18]

Fig. 6 – Connection buckling with restrainer end hinging

The common characteristic of these specimens that failed before their calculated strength was reached (i.e. $P_{exp} < P_{uc}$) is a short insert length resulting in low moment transfer capacity at the restrainer end, combined with a gusset or adjacent framing offering little out-of-plane rotational stiffness. The buckling mode shapes are characterized by either rigid body rotation of the gusset, or a combined rotational-flexural mode shape, corresponding to Concept A shown in Fig.3.

Restrainer end moment transfer details have traditionally been justified indirectly by demonstrating overall stability for subassembly tests of a representative set of specimens, whether for braces with a cruciform insert or collar. However, since stability is a function of 1) *restrainer end moment continuity*, 2) *gusset flexural stiffness*, and 3) *gusset and adjacent framing rotational stiffness*, subassembly testing with additional artificial stiffness in one or more of these aspects is insufficient to validate system stability. Given that a wide range of creative gusset and framing configurations are observed in practice, whether for aesthetic, functional or other structural reasons, it does not seem practical or economic to robustly verify all configurations through testing, and so a robust analytical approach is desirable.

An analytical and experimental study by Matsui and Takeuchi [11] investigated the required insert ratio for cruciform transition zones, proposing formulas for the rotational stiffness and concluding that $L_{in}/D_n \approx 2$ is required for full continuity. This is a greater embedment length than can often be found in practice, which can be in the order of $L_{in}/D_n \approx 1.0$. Note that while a low insert ratio does not necessarily imply susceptibility to connection buckling, it will drastically increase the stiffness requirements for the rest of the system.

Gusset buckling is frequently assessed using an idealized effective length factor k_e based on research of concentric braced frame gussets. Unfortunately, the potential for restrainer end hinging and increased initial imperfections due to the debonding gap often results in a far lower inelastic buckling capacity.

A companion paper by Takeuchi *et al.* [13] reviews the various buckling design criteria proposed specifically for BRBs and a unified method is introduced considering the restrainer end moment transfer, adjacent framing, gusset and connection stiffness, and out-of-plane drift. Remarkably, realistic cases with low restrainer end moment transfer capacity, and with reasonably flexible adjacent framing can result in exceptionally large equivalent effective length factors ($k_e \approx 2$~8) with respect to the full connection length.

3.2 Buckling with gusset hinging

Buckling can also be initiated by gusset inelasticity, with the inelastic buckling mode resembling a sway mechanism and with the restrainer end hinge potentially following shortly thereafter.

Tsai and Hsiao [19] tested a 3 story frame in-plane with transverse beams, composite slab and various types of BRBs arranged in a chevron configuration. Buckling was observed during Phase 1 testing at the 1st and 3rd story gussets, despite complying with US AISC guidelines (Whitmore width, maximum Thornton length and k_e=0.65). The 1st story buckling occurred at a beam/column joint with Gusset Type A early in the testing regime, while the 3rd story buckling occurred at a beam connection with Gusset Type A at 2% story drift. No instabilities were observed during later Phase 2 testing, which employed the same gussets but with additional stiffeners welded in a Type C configuration. The buckling mechanisms are shown in Fig.7:

(a)	(b)
$P_y \approx 560$kN	$P_y \approx 1250$kN
$P_{uc} \approx 760$kN	$P_{uc} \approx 1710$kN
$P_{exp} = 755$kN	$P_{exp} = 805$kN
(a) 3BRB, Tsai & Hsiao [19]	(b) 1BRB, Tsai & Hsiao [19]

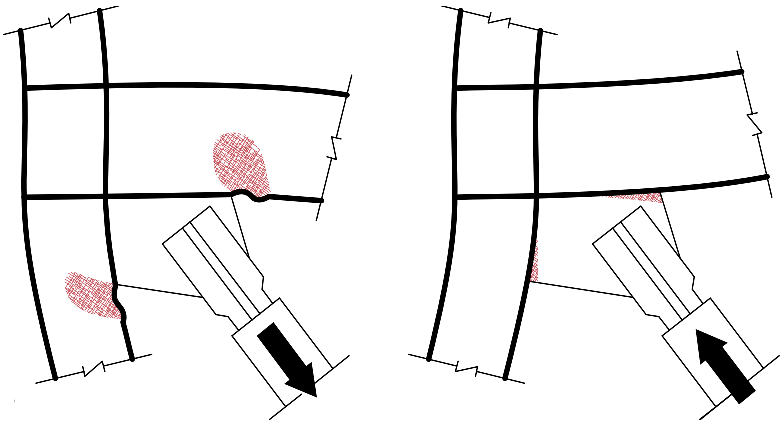
Fig. 7 – Connection buckling initiated by gusset sway

Though this mechanism resembles traditional gusset buckling, the flexible moment transfer mechanism at BRB restrainer ends can lead to lower capacities than predicted by methods developed for fully continuous concentric braced frames, such as Muir and Thorton [20]. Westeneng *et al.* [21] provide a comparison of this buckling mode and show that the gusset plate effective length factor, k_e, depends on the restrainer stiffness and restrainer end continuity, but can exceed 3 in realistic situations. The potential for inelasticity in the gusset is also considered in the unified global buckling calculation proposed by Takeuchi *et al.* [13].

4. Beam/column damage and connection weld fracture

Gussets tend to act as a rigid haunch, stiffening the beam-column connection in the in-plane direction with the effect of shifting the plastic hinge zone away from the beam end and amplifying gusset demands. Frame pinching and opening effects have been noted to cause concentrated buckling, yielding and fracture in adjacent beams and columns, and gusset weld fractures, respectively [18,22-25].

Typical damage patterns during brace tension and compression cycles are shown in Fig.8:



(a) Frame pinching (b) Frame opening

Fig. 8 – Frame effects due to in-plane drift

Palmer *et al.* [18] tested 6 bolted BRBs in a single diagonal frame configuration with 19mm gussets. These gussets were attached with a bolted end plate or with 12mm fillet welds on both sides to a welded moment frame. The beam and columns had 9mm and 11mm webs, respectively, with no stiffeners. An additional 4 pinned specimens were tested in a 2 storey 3D frame in a single diagonal configuration. The purpose of the test was to investigate gusset demands under in-plane drift, varying the taper (0°, 15°, 35°) and connection type. Significant yielding was observed in the adjacent framing elements and gussets in all tests. Gusset weld fracture and local buckling of the beam flange was observed in the single frame tests with the local buckling creating a softened condition that led to out-of-plane connection instability at ~2% drift.

Uriz *et al.* [22] reviewed testing of 3 bolted specimens in chevron and single diagonal frame configurations, with 25mm rectangular gussets attached with full-penetration butt welds to a welded moment frame with stiffeners, 15mm beam webs and 21mm column webs. The purpose of this testing was to determine the performance of the gussets and overall frame under in-plane drift. Significant yielding was observed in the adjacent framing elements and gusset during all tests. In the two single diagonal brace tests, cracks formed at the gusset/framing weld and beam fracture occurred at the gusset tip. This precipitated out-of-plane connection instability at 2.25% drift.

Lin *et al.* [23] reviewed testing of a 3 story frame in a chevron configuration with 15mm non-tapered gussets attached with fillet welds to a welded moment frame with 11 to 13mm beam webs and 16mm column webs. The purpose was to confirm the performance of various BRB types and connection performance. Though gussets were designed according to normal AISC design provisions, the gusset-to-flange welds partially fractured prior to exhausting the BRB ductility capacity.

Kasai *et al.* [25] tested 10 specimens in a subassembly with 9 or 12mm webs, 16 or 22mm flanges and 9 or 19mm non-tapered gussets both with (Type C) and without (Type B) edge stiffeners. At 0.7%~2% drift, extensive web yielding, local buckling and fracture was observed for cases (#4 & 5) where the web was thinner than the gusset, and a gusset cracked when it was sized thinner than the web (#6).

Final damage states and corresponding interstory drift ratios of selected recent tests are shown in Fig.9:

2.2~3.5% drift — (a) Palmer [18]
3.2% drift — (b) Lin [23]
3.3% drift — (c) Lin [23]
0.7~2% drift — (d) Kasai [25]

Fig. 9 – Damage to adjacent framing and gussets due to in-plane drift

In-plane frame action imposes compatibility demands on the gusset, which if not accounted for in design, results in yielding, local buckling, and fracture of the adjacent elements and gussets. Extensive yielding of the beams and columns may degrade the framing stiffness sufficiently to initiate out-of-plane BRB instability and global failure. Damage to the beams and columns also hinders post-earthquake repair.

This can be addressed by strengthening the members and designing for the compatibility demands, or by using special details that release the compatibility demands. To directly accommodate the frame rotation compatibility demands, Palmer *et al.* [18] and Kasai *et al.* [25] suggested to design the gusset-to-flange welds for gusset strength and to balance the gusset and framing web thickness. Lin *et al.* [23], Chou *et al.*

[24] and Muir and Thornton [20] proposed modifications to the Uniform Force Method that account for both axial and framing rotation-induced forces.

Modified details include reducing gusset fixity or introducing a hinge in the beam at the end of the gusset. Palmer *et al.* [18] studied the effect of tapering the gusset to reduce the length of the weld to the beam/column flanges and observed that this reduced or delayed, but did not eliminate, damage to the adjacent framing elements. Berman and Bruneau [26] proposed to attach the gusset only to the beam and then to design for the additional eccentricities. Wigle and Fahnestock [27] and Prinz *et al.* [28] proposed to incorporate a hinge in the beam at the end of the gusset, releasing frame rotations. This beam pin detail is used in practice, an example from a recent project in New Zealand is depicted below in Figure 10. Note that all of these approaches reduce frame action, which of course reduces the structure's post-yield stiffness.

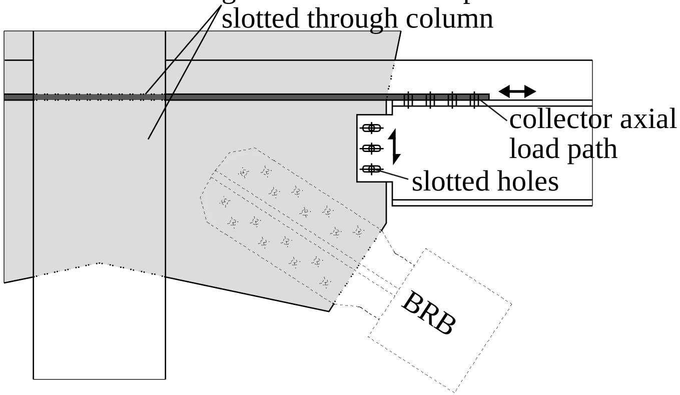
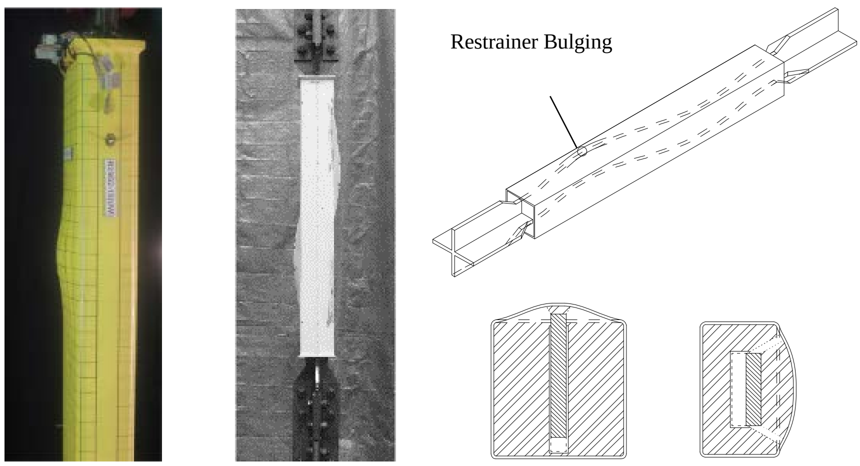


Fig. 10 – Beam splice detail releasing frame compatibility demands on gusset

5. Restrainer wall bulging

Slender, rectangular restrainer tubes with rectangular cores have been observed to fail prematurely in the bulging mechanism shown in Fig.11c [29-31]. This phenomenon is induced by higher mode inelastic buckling of the core, which forms in the small gap of the debonding zone and additional gap opened during tensile cycles due to Poisson effects. The immediate cause of failure is thin restrainer walls with insufficient flexural capacity to resist the buckled waveform's perpendicular force. Once bulging commences, it typically results in severe compressive stiffness degradation and concentrated core strain amplification. However, the residual stiffness of the deformed restrainer is occasionally sufficient to enable the BRB to continue dissipating energy without a significant loss of compressive stiffness [31]. The restrainer can bulge either perpendicular to, or parallel to, the axis of the rectangular core plate.



(a) Lin *et al.* [31] (b) Takeuchi *et al.* [30] (c) Bulging mechanism
modified from [30]

Fig. 11 – Bulging failure of restrainer casing

Takeuchi *et al.* [29,30] conducted cyclic uniaxial testing of 15 mortar filled specimens with square and circular casings. No failures were observed for those with circular casings, but bulging and subsequent severe loss of stiffness or fracture was observed a number specimen with slender square casings. Susceptibility was found to be sensitive to the casing wall slenderness ratio (width/thickness), core dimensions, core to restrainer mortar thickness, debonding gap and previous tensile cycles.

Lin *et al.* [31] conducted cyclic uniaxial testing of 22 mortar filled specimens, with bulging observed in 13 specimens and 8 of those immediately losing compressive capacity. In addition to the parameters listed above, the sensitivity to mortar strength and loading protocol was also investigated. Higher strength mortar was found to slightly delay the failure mode, while cyclic loading history also was found to have some effect. Large strains increase the strain-hardened perpendicular force and extent of mortar cracking, while large cycle counts at just below the critical strain hardened force may mask bulging failure, with the core first fracturing due to low cycle fatigue.

Tests from Takeuchi *et al.* [29,30] and Lin *et al.* [31] where restrainer bulging was observed are listed in Table 3, with the bulging mechanisms of typical specimens shown in Fig.11a and Fig.11b:

Table 3 – Bulging failures [29-31]

	Specimen	Restrainer $D_r x B_r x t_r$ [1] [mm]	Slenderness (B_r/t_r)	Gap [mm]	Mortar Thickness [mm]	Yield Force [kN]	Max Force [kN]	Approx. Bulging Strain
[29]	RY65	150x150x2.3	65	1	7	543	962	2.0%
	RrY125	100x100x0.8	125	1	3	298	432	3.0%
	RrY125M	100x100x0.8	125	1	14	300	454	3.0%
	RrY63	100x100x1.6	63	1	2	298	509	3.0%
[30]	RY65M25	150x150x2.3	65	1	25	507	902	3.0%
	RY76M37	175x175x2.3	76	1	37	507	914	3.0%
[31]	W16G2-85-IN	150x250x9	28	2	56	499	886	3.5%
	W16G2-95-IN	150x250x9	28	2	56	558	885	3.5%
	R25G2-200-IN	200x400x12	33	2	74	1965	2745	3.0%
	F25G2-160-IN	200x400x12	33	2	74	1572	2134	3.5%
	F25G4-160-IN	200x400x12	33	4	72	1572	1920	3.0%
	R25G2-250-IN[2]	200x400x12	33	2	74	2456	3432	3.0%
	C40G2-160-IN	150x350x12	29	2	41	2547	3767	3.5%
	C25G2-250-IN	150x350x12	29	2	49	2456	3402	3.0%
	F25G2-160-DN	200x400x12	33	2	74	1416	2155	3.0%
	F25G2-160-IH[2]	200x400x12	33	2	74	1416	2216	3.5%
	R25G2-200-DN[2]	200x400x12	33	2	74	1770	2863	3.0%
	R40G2-120-IN[2]	200x400x12	33	2	66	1829	2838	3.5%
	R40G2-200-SN[2]	200x400x12	33	2	66	3048	4545	3.0%

[1] Dimensions orientated such that bulging observed in restrainer wall with dimension $B_r x t_r$

[2] Maintained significant compressive stiffness following bulging

The common characteristic of these specimens is a slender rectangular restrainer tube and rectangular core. Circular tubes are generally not susceptible to this mechanism due to hoop tension, and cruciform cores better distribute the perpendicular force, with these braces tending not to experience this mode of failure.

While proportioning rules can easily be described qualitatively, reliable calculation of susceptibility to bulging is a function of a number of parameters, preventing use of simple slenderness limits. Some suppliers have relied upon extensive uniaxial testing, while Takeuchi *et al.* [30] and Lin *et al.* [31] have developed simple formulas based on first principals to directly predict this failure mechanism analytically.

6. Conclusion

Key failure mechanisms of BRBs have been reviewed, highlighting a number of design considerations and best practice in analysing and mitigating these phenomena.

1) Global stability requires consideration of the out-of-plane stiffness of the adjacent framing, gusset and connection zone, and restrainer end continuity and hence moment transfer capacity, which is reduced by out-of-plane drift. Full fixity assumptions at the gusset base are not conservative, especially for details with substantial out-of-plane rotational flexibility, such as many common chevron beam connections. Takeuchi *et al.* [13] proposes a unified stability design method considering these effects.

2) Connection buckling occurs in a sway mode and using k_e<1.0 is generally non-conservative. In certain cases with significant out-of-plane flexibility and restrainer end hinging, buckling can occur at loads corresponding to an equivalent k_e>2.0. Hinging at the restrainer ends occurs when low moment transfer capacity is provided, which is associated with cruciform neck insert ratios of $L_{in}/D_n < 2.0$ (refer Fig.5) [11]. The method proposed in Takeuchi *et al.* [13] includes connection buckling modes.

3) Gusset weld fracture and damage to adjacent framing members has been observed due to frame opening or pinching compatibility effects during in-plane drift. This damage increases post-event repair costs and in some cases out-of-plane instability or direct failure has been observed. Either the additional demands should be directly considered in design [18,20,23-25], or details employed to release the in-plane drift induced compatibility demands [26-28].

4) Restrainer bulging and subsequent severe loss of compressive stiffness can occur in slender rectangular restrainer casings due to higher mode buckling of rectangular cores. Bulging is sensitive to precise core and restrainer dimensions, as well as the debonding gap and strain history. Takeuchi *et al.* [30] and Lin *et al.* [31] provide analytical methods to predict this mechanism.

Current codified design practice generally ignores or non-conservatively accounts for these mechanisms and the widely used testing procedures of AISC 341 do not necessarily guarantee satisfactory performance. However, analytical methods backed by rigorous empirical results have recently been developed to design for each of these mechanisms. Demonstrated by several full-scale, 3D tests and performance during 2011 Tohoku earthquake, well designed BRB frames can be a robust, high performance and efficient seismic system.

7. References

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