Key Engineering Materials
ISSN: 1662-9795, Vol. 763, pp 50-60
doi:10.4028/www.scientific.net/KEM.763.50

Buckling-Restrained Brace: History, Design and Applications

Toru Takeuchi[1,a]

[1]Tokyo Institute of Technology, 2-12-1-M1-29 Ookayama Meguro-ku Tokyo Japan

[a]takeuchi.t.ab@m.titech.ac.jp

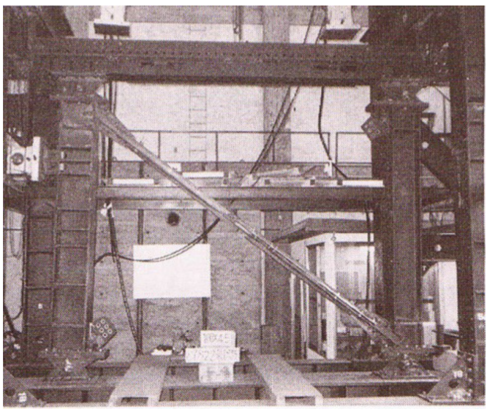
Keywords: Buckling-Restrained Brace, Research History, Buckling, Energy Dissipation

Abstract. Buckling-restrained braces (BRBs), which were first applied in 1989 in Japan, are now widely used worldwide as ductile seismic-proof members in seismic zones, such as those in Japan, USA, Taiwan, China, Turkey, and New Zealand. Although the design procedures of BRBs and their applications are described in the design codes and recommendations of several countries, they do not necessarily cover all the required aspects. Moreover, various new types of BRBs are still under investigation by many researchers. In this paper, the early history of BRB research and development and state-of-the-art views on the items required to design BRBs for obtaining stable hysteresis are briefly overviewed. This is followed by a summary of various representative application concepts and up-to-date investigations.

Introduction

The buckling-restrained brace (BRB) is a seismic device consisting of an axially yielding core and axially decoupled restraining mechanism, which suppresses the overall buckling. The hysteretic characteristics are stable and nearly symmetric once the full cross section of the core has yielded, differing only slightly from the base material hysteresis. Because buckling is restrained, no associated degradation should be appeared during the compression cycles. For this unique behavior, BRBs can be modeled using truss elements and uniaxial material hysteresis rules, assuming strain is distributed along the full plastic core length.

The basic concepts of buckling-restrained braces appeared from the 1970s, when limited successes were reported by several researchers in Japan and India [1–3]. The first practical BRB was achieved by Saeki, Wada, et al. [4, 5] in 1988. They employed rectangular steel tubes with in-filled mortar for the restrainer, and determined the optimal debonding material specifications to obtain stable and symmetric hysteretic behavior. In addition, the basic theory to design the restrainer was established and the first project application soon followed. In 1989, these BRBs (unbonded braces) were applied to two 10- and 15-story steel frame office buildings, in the first project to use BRBs [6]. BRBs increased in popularity and other configurations soon followed, notably the all steel tube-in-tube type.



(a) Cyclic loading test in 1987 [4,5]

(b) The first BRBF application in 1989 [6]

Fig.1 Early development of BRBs in Japan.

Through the 1990s, BRBs were used in approximately 160 buildings in Japan. In July 1995, the concept of "damage tolerant structure" was proposed by Wada, Iwata, et al. [7], which uses BRBs as energy dissipating elasto-plastic dampers within an elastic main frame. The AIJ design recommendations included BRBs design guidelines for the first time in 1996 [8].

Collaboration with researchers in US soon led to the first international application, with the construction of a building at UC Davis in 1998, followed by an experiment at UC Berkeley in 2000 [9]. Numerous other buildings with BRBs were soon constructed throughout California, including some in seismic retrofit applications. In 2002, a design guidance for the buckling-restrained braced frame (BRBF) was included in the Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-05) [10]. During these early years of technology transfer to international markets, a series of symposiums on passively controlled structures were held at Tokyo Tech, sharing code developments, BRB designs, and novel applications [11]. Through the following decade, BRBs increased in popularity in numerous countries, from Taiwan in the early 2000s [12] to the recent implementations in New Zealand as part of the Christchurch rebuild. BRBs are now widely known in seismic areas throughout the world, with research ongoing in countries such as Japan, Taiwan, China, USA, Canada, Turkey, Iran, Italy, Romania, New Zealand, and Chile.

Requirements for Stable Hysteresis

In general, the BRB must be designed for strength and stability, considering both its local and global behavior, as shown in Fig. 2.

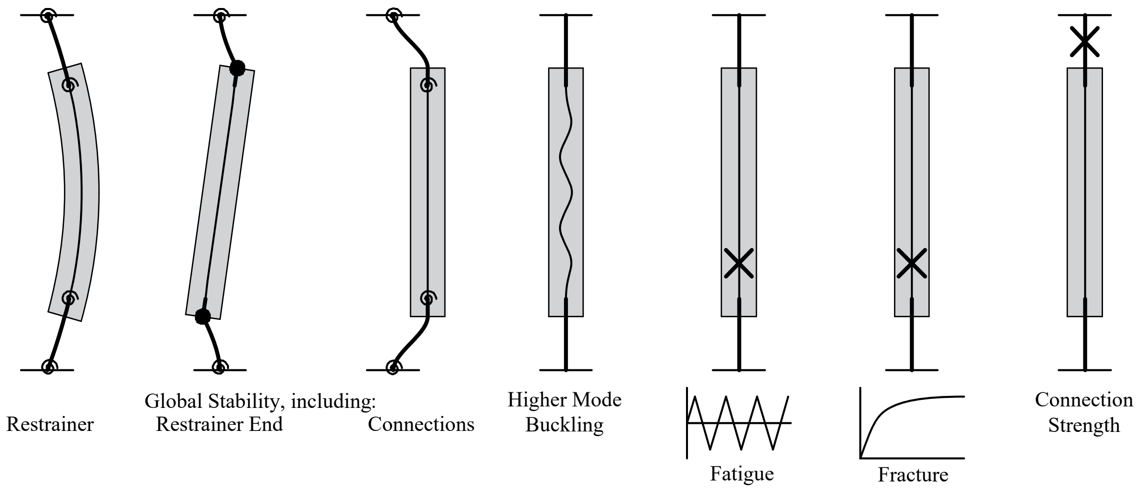


Fig.2 BRB stability and strength.

To obtain a stable hysteresis, the following design conditions shall be basically satisfied [8].

1. The *Restrainer* successfully suppresses first-mode flexural buckling of the core
2. The *Debonding mechanism* decouples axial demands and allows for Poisson effects of the core
3. The *Restrainer wall bulging* owing to higher mode buckling is suppressed
4. *Global out-of-plane stability* is ensured, including connections
5. *Low-cycle fatigue capacity* is sufficient for the expected demands

For designing the restrainer to suppress the global buckling of the core, the restrainer flexural yield strength $M^B{}_y$ should satisfy [13]:

$$M^B = \frac{N_{cu} a_c}{1 - N_{cu} / N_{cr}^E} = \frac{N_{cu}\left(a + 2s_r + e\right)}{1 - N_{cu} / N_{cr}^E} \le M_y^B \tag{1}$$

where a : fabrication imperfection of core and/or brace, s_r : clearance or thickness of debonding material (per face), e : eccentricity of the axial force, $M^B{}_y$: flexural strength of the restrainer, $N_{cu} = {}_d\,\alpha N_y$: core yield strength amplified by overstrength and strain hardening, and $N^E{}_{cr}$: Euler buckling strength of the restrainer, which is given by:

$$N_{cr}^{E} = \frac{\pi^2 EI_B}{l_B^{\ 2}} \tag{2}$$

For the case of initial imperfections $a_c/l_B \leq 1/500$, a relatively slender restrainer with $l_B/D_r > 20$ and with an overall safety factor of $_e\alpha \geq 1.5$, Eq.(1) can be reduced to Eq.(3).

$$N_{cr}^{B} = \frac{\pi^2 EI_B}{l_B^{\ 2}} > {}_e\alpha N_{cu} \tag{3}$$

The purpose of the debonding layer is to prevent significant compressive loads from being passed to the restrainer, preventing it from buckling and ensuring a balance hysteresis. This is achieved by introducing a low friction interface and by accommodating the Poisson effect expansion of the core under compressive loads, either through the provision of a suitable gap or a compressible material or through elastic deformation of the restrainer material. However, the gap must be closely controlled as it is directly related to the higher mode buckling amplitude.

$$s_r \geq \frac{\nu_p \varepsilon_{max} B_c}{2} \text{ (per face)} \tag{4}$$

where s_r : appropriate clearance, ν_p : the plastic Poisson ratio (= 0.5), ε_{max} : maximum expected tensile strain, and B_c : core width.

Local Bulging Failure

The compressible debonding layer between the steel core and restrainer provides a space for the flat steel core to form high mode buckling waves when the BRB is under compression. An in-plane or out-of-plane local bulging failure would occur if the steel tube strength is insufficient to sustain the in-plane or out-of-plane outward force. To avoid local bulging failure, the following criteria should be satisfied [14–18, 44].

$$DCR_s = \frac{P_{d,s}}{P_{c,s}} = \frac{(D_r - t_c)}{(2D_r - t_c)t_r^2\sigma_{ry}} \cdot \frac{4N_{cu}\left(2s_{rs} + \nu_p B_c \varepsilon_t\right)}{l_{p,s}} < 1.0 \tag{5}$$

$$DCR_w = \frac{P_{d,w}}{P_{c,w}} = \frac{(B_r - B_c)}{(2B_r - B_c)t_r^2\sigma_{ry}} \cdot \frac{4N_{cu}\left(2s_{rw} + \nu_p t_c \varepsilon_t\right)}{l_{p,w}} < 1.0 \tag{6}$$

Comparisons between the test results and proposed equations are shown in Fig. 3. The effects of the steel tube thickness (t_r), debonding layer thickness (s_{rs} and s_{rw}), loading sequences, and in-filled mortar compressive strength on the test results are discussed in the following sections.

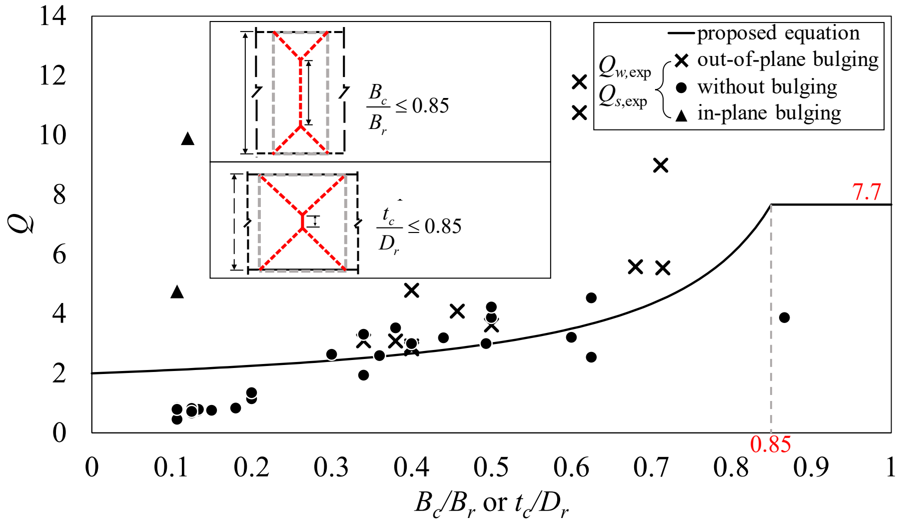


Fig.3 Comparisons between test results and proposed equations.[44]

When the in-filled mortar in the restrainer does not have enough strength, it could be crushed by the acting outward forces. If the contact surface is B_c long and l_c wide, the criterion could be expressed as in Eq. (7):

$$\frac{P_{d,w}}{l_c B_c} < f_c' \quad (7)$$

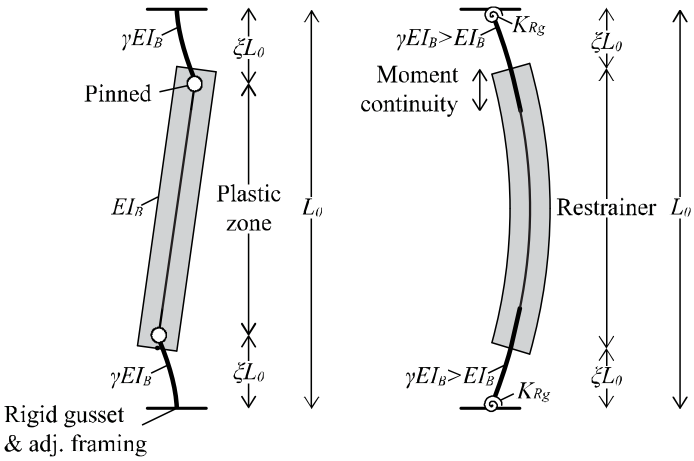
where f'_c is the allowable compressive strength of the in-filled mortar. Although the value of l_c requires additional research, it can be estimated generally as $l_c \approx t_c$.

Global Instability Including Connections

For preventing global instability including connections, two stability design concepts were proposed in the 2009 AIJ *Recommendations for Stability Design of Steel Structures* [8], and are shown in Fig. 4.

(1) *Cantilever Connection Concept*: Effectively rigid adjacent framing and gussets are provided, so that the restrainer end continuity can be neglected. Stability is ensured by designing the connection zone as a simple cantilever (Fig. 4 (a)) [19–21].

(2) *Restrainer Continuity Concept*: Full restrainer end moment transfer capacity is provided, permitting more flexible gusset or adjacent framing details. The buckling analysis is more complex, with the critical hinge located at either the neck or gusset (Fig. 4 (b)) [22, 23].



(a) Cantilever Connection (b) Restrainer Continuity

Fig.4 BRB stability condition concepts.[8]

The *Cantilever Connection Concept* (a) primarily relies on the gusset and adjacent framing rotational stiffness. The gusset rotational stiffness K_{Rg} is largely governed by the stiffener topology (Fig. 5), and therefore, either gussets type C or D can be employed if the *Cantilever Connection Concept* is selected. However, if full-depth stiffeners are omitted (gussets type A or B), the connection stiffness rapidly decreases, with out-of-plane rotation concentrating at the gusset. This has a dramatic effect on the elastic buckling load, which can easily be less than 30% of the pure cantilever-buckling load.

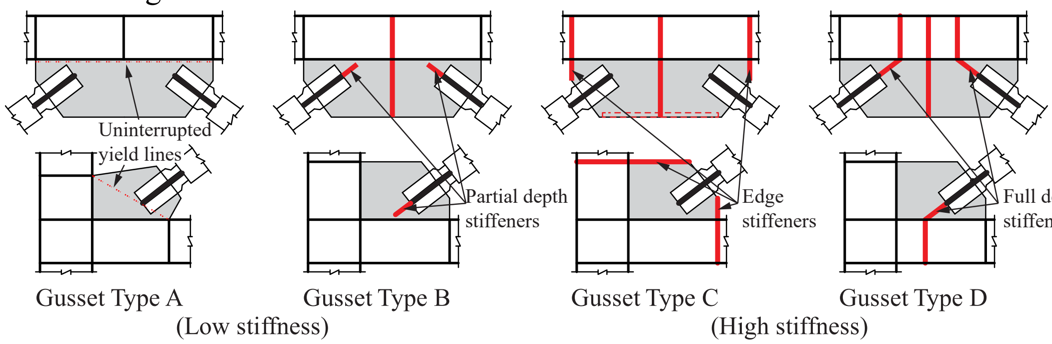


Fig.5 Gusset plate types and out-of-plane stiffness.

The *Restrainer Continuity Concept* described in Fig. 4 (b) is based on the analysis of the full BRB with continuity provided at the restrainer ends. Although several design equations have been proposed, the Takeuchi's proposal in 2014 [22] provides the most general criteria.

$$N_{lim1} = \frac{\left(M_p^r - M_0^r\right)/a_r + N_{cr}^r}{\left(M_p^r - M_0^r\right)/a_r N_{cr}^B + 1} > N_{cu} \tag{8}$$

where N^{r}_{cr} is the elasto-plastic buckling load of N^{R}_{cr}, and $M_p^r - M_0^r$ should be taken as zero if the difference is negative. The criteria when the gusset produces plastic hinges are given as follows:

$$N_{lim2} = \frac{\left[(1-2\xi)M_p^g + M_p^r - 2M_0^r\right]/a_r}{\left[(1-2\xi)M_p^g + M_p^r - 2M_0^r\right]/(a_r N_{cr}^B) + 1} > N_{cu} \tag{9}$$

where M_p^g is the plastic bending strength of the gusset plate including the axial force effect, and $(1-2\xi)M_p^g - M_0^r$ or $M_p^r - M_0^r$ should be taken as zero if the difference is negative. The minimum value of N_{lim1} and N_{lim2} is defined as the stability limit N_{lim}, which should be smaller than N_{cu}.

Cumulative Deformation Capacity until Fracture

The cumulative deformation capacity of a BRB under constant axial displacement amplitude can be roughly modeled following the Manson-Cofin's rule. Its performance is reduced compared to that of the steel material, because of uneven plastic strain distributions in the core plates caused by the local wave generated within the debonding gap (Fig. 6). Therefore, it should be noted that the low-cycle fatigue changes depending on the debonding gap and their fabrication tolerances [25].

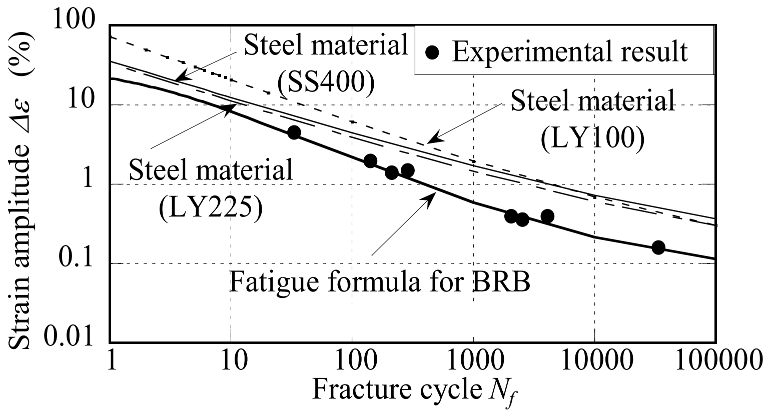


Fig.6 Low-cycle fatigue capacity example for BRB and steel material.

The fracture criteria under a random amplitude response are normally evaluated through the Miner's rule using these fatigue curves. Takeuchi et al. also proposed the following criteria using averaged amplitudes, which do not require detailed strain time-histories [26]:

$$\chi = \frac{1}{\dfrac{\alpha_S}{\chi_{SO}} + \dfrac{(1-\alpha_S)}{4}\left\{\dfrac{\overline{\Delta\varepsilon_{ph}}^{(1+m_2)}}{C}\right\}^{-\frac{1}{m_2}}} \tag{10}$$

where $\overline{\Delta\varepsilon_{ph}}$ = half of the average plastic strain amplitude. Eq. (10) gives the same criteria as the Miner's rule when the exponential value of the fatigue curve m_2=1 [25].

Performance Test Specification for BRB

Although the axial yielding mechanism of BRBs is conceptually simple, its performance depends on the precise detailing of the debonding mechanism and restrainer, and is sensitive to fabrication quality. To ensure that a BRB will perform as intended, most jurisdictions require physical testing, either as part of a supplier prequalification or on a project-specific basis. It is important that the test specimens be fabricated by the appointed manufacturer, be of similar proportions, and use the same details as those used in the design. The detailed specifications are described in AISC 341-16 [10] and the Building Center of Japan (BCJ), whose testing protocols are summarized in tables. The detailed requirements of testing are described in [44].

BRBF Applications

Various structural design concepts using BRBs have been proposed and realized over these 30 years. Some of them are introduced below.

1) Damage tolerant concept

In 1992, Wada et al. [27] proposed the concept of "damage tolerant structures" where energy dissipation is concentrated in special members designated as "damage fuses" and the main structure is kept safe to carry gravity loads (Fig. 7). An early example of a damage tolerant structure is the Triton Square Project, a 40-story (180 m) office building located in Tokyo. A typical floor plan is 50 m x 50 m and the frame consists of HT780 columns, HT590 beams, and LY100 BRBs on all four sides. While the BRB layout introduces some inefficiencies owing to the indirect connection, the low yield strength ensured a sufficient yield drift angle. Optimal distribution methods of BRBs using equivalent linearization techniques were also developed and applied in these damage tolerant designs [28, 29].

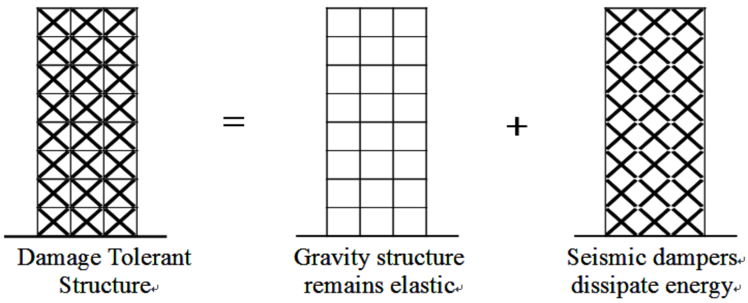


Fig.7 Concept of damage tolerant structure.[27]

Fig.8 Triton Square, 1992.

2) Retrofit using BRBs

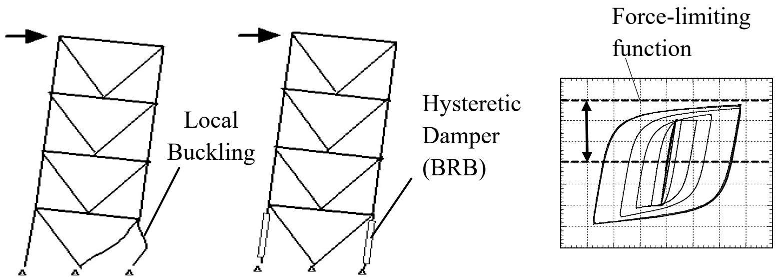
BRBs have several desirable characteristics that frequently receive attention for retrofit projects. One of the typical retrofit strategies for non-ductile moment frames is to install BRBs as bracing elements along the perimeter, as either an external frame or in-plane with the existing one. However, such retrofits are often difficult to implement while maintaining continuous occupation, and frequently will have a negative effect on the building aesthetics. At this point, it should be recognized that façades have various functions; they are not only a suitable location for seismic reinforcement, but also affect energy efficiency and architectural appearance. To resolve these competing functions, the concept of "integrated façade engineering" has been proposed, combining the structural retrofit, façade design, and environmental design, and including improvements on seismic performance using seismic energy dissipation devices as BRBs (Fig. 9) [30].

Fig.9 Retrofit of existing RC building with BRBs (before and after).[30]

There are various proposals for the connections attaching BRBs to RC frames [33, 34]. It is recommended to insert elastic steel frames together with BRBs for soft and weak RC structures, not only for reliable sheer force transition but also for providing self-centering and damage distribution functions [31, 32].

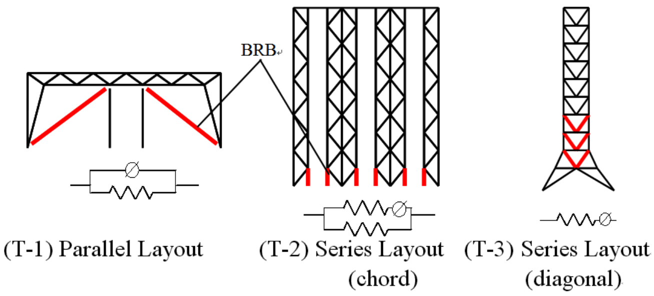
3) Applications to trusses and spatial structures

Two key challenges arise when applying BRBs to spatial structures: 1) these are often so light that the required core size is extremely small, and 2) it can be a challenge to find attachment positions with sufficient relative displacements to be efficient. Fig. 10 (a) shows a conventional truss with the capacity determined by buckling of the column or brace members. A basic strategy to improve the seismic response is shown in Fig. 10 (b), where the critical members are replaced with BRBs, improving the collapse mechanism, increasing the energy dissipation capacity, and protecting the remaining compression members with the BRB's force-limiting function [35]. Typical BRB layouts for truss structures and latticed roof structures are shown in Figs. 11 and 12 [36].



(a) Conventional (b) Response Controlled

Figure 10 Response Control for Truss Structures.[35]



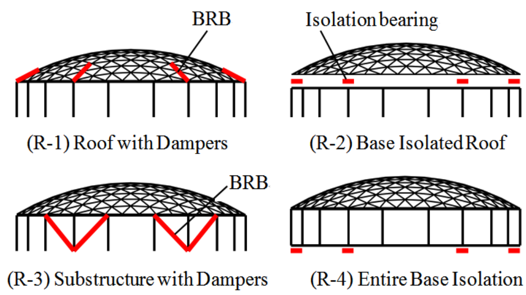


Fig.11 BRB layout for truss structures.[36] Fig.12 BRB layout for spatial structures.[36]

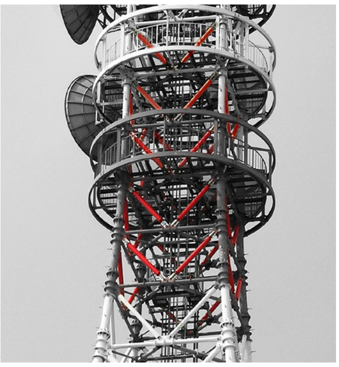


Fig.13 Retrofit of communication tower.[37]

Fig.14 Toyota stadium.[37]

Fig. 13 shows an example of BRBs used for a retrofit program of communication towers that had been constructed in Japan in the 1970s. Compared to the strength-based retrofit, replacing critical diagonal members with BRBs is a more economical and effective way to save other existing members. Fig.14 is a sample of BRBs applied to the supporting structure of a spatial structure. Although raised roofs produce vertical excitation even against a horizontal earthquake input, the energy-dissipation provided by BRBs is known to be effective in reducing such roof response, and thus, BRBs have been used for the retrofit of existing gymnasiums [38]. Similarly, bridge applications have also increased in recent years. In Japan, BRBs are frequently used to retrofit steel arch bridges [38, 39].

4) Spine frame concept

One of the relatively recent applications of BRBs is their use as part of a rocking or spine frame, alternatively known as a "strong-back system", or "mast frame." When BRBs are used as the sole lateral force resisting system, their low post-yield stiffness may result in damage or in the residual drift concentrating at one level, even if the capacities are relatively well balanced over the height of the structure [41]. Such damages were observed in the Great Hanshin Earthquake in 1995. To avoid this risk, numerous researchers and practitioners have proposed spine frame systems featuring various combinations of damper, rocking, and/or restoring components. Taga et al. [40] distributed BRBs along the vertical elastic spine composed of a strongly braced frame, which is named as "dual spine" (Fig. 15). A similar concept was proposed by Lai, Mahin et al. [42], who named it the "strong-back system," and which has been implemented in a low-rise structure in California.

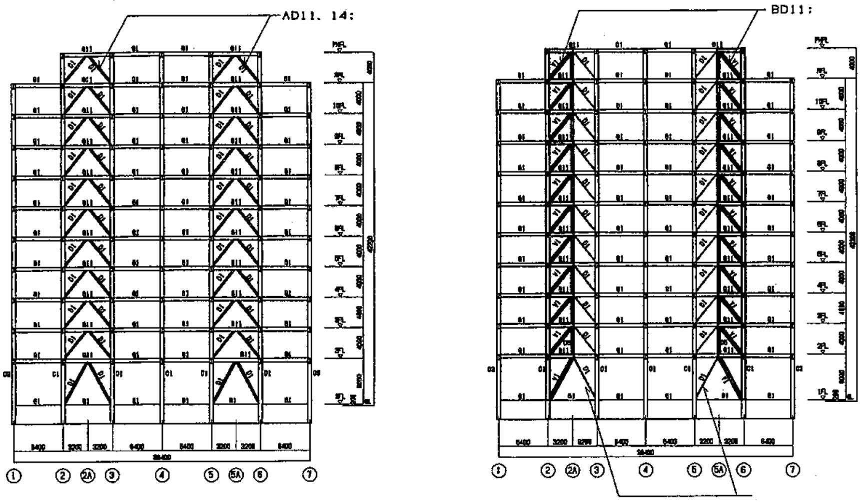


Fig.15 Dual Spine (Strongback) System.[40]

An alternative rocking frame concept arranges the BRBs as the first-story column elements, which then are called as buckling restrained columns (BRCs). This creates an uplifting "Controlled Rocking-frame [41]" with PT-wires, the rocking frame acts as a spine to avoid damage at soft stories. Similar to the previous concept, non-uplifting system avoiding the need for complicated uplift details can be composed, where the restoring force is provided by either an envelope moment frame or gravity. This system was introduced earlier, in Fig. 10, and was implemented in a 5-story laboratory building at Tokyo Institute of Technology, completed in 2014, which is shown in Figs. 16–17 [43].

Fig.16 Material research building.[43]

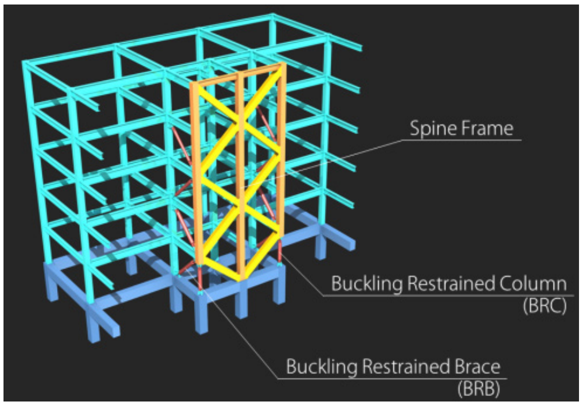


Fig.17 Structural system.[43]

Conclusive Summary

In this paper, the early history and general key factors of BRBs are briefly introduced, followed by representative application concepts. All the detailed theories and information are described in the related recent publications [44].

References

[1] Takeda T, Takemoto Y, Furuya Y: An experimental study on moment frame with steel braces Part 3, *Annual Meeting AIJ*, 1972; 47:1389–1390. (*in Japanese*)

[2] Wakabayashi M, et al. : Experimental studies on Precast concrete wall including un-bonded braces under cyclic loading Part 1, *Annual Meeting AIJ*, 1973; 48:1041–1042. (*in Japanese*)

[3] Mochizuki S, et al.: An experimental study on the buckling behaviour of steel braces with concentric axial forces Part 1, *Annual Meeting AIJ*, 1979; 54:1623–1624. (*in Japanese*)

[4] Fujimoto M, Wada A, Saeki E, Watanabe A, Hitomi Y: A study on the unbounded brace encased in buckling-restraining concrete and steel tube, *Journal of Structural Engineering*, 1988; 34B:249–258. (*in Japanese*)

[5] Watanabe A, Hitomi Y, Saeki E, Wada A, Fujimoto M: Properties of brace encased in buckling-restraining concrete and steel tube, *Proc. 9WCEE*, 1988; IV:719–724.

[6] Fujimoto M, Wada A, Saeki E, Takeuchi T, Watanabe A: Development of unbonded brace, *Querterly Column*, 1990; (115):91–96.

[7] Wada A, Iwata M, Huang YH: Seismic design trend of tall building after the Kobe earthquake, *Proc. Int. Post-SMiRT Conf. Seminar on Seismic Isolation, Passive Energy Dissipation, and Control of Vibrations of Structures*, *Taormina, Italy*; 1997:25–27

[8] Architectural Institute of Japan: *Recommendations for Stability Design of Steel Structures*, 1996, 2009, 2017

[9] Clark P, Aiken I, Kasai K, Ko E, Kimura I: Design Procedure for buildings incorporating hysteretic damping devices, *68th annual convention SEAOC, CA*; 1999:355–371.

[10] AISC: *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-05), 2002, (ANSI/AISC 341-10), 2010 , (ANSI/AISC 341-16), 2016

[11] Tokyo Institute of Technology: *Proceedings of passively controlled structures symposium*; 2000, 2001, 2002, 2004.

[12] Tsai KC, Lai JW, Hwang YC , Lin SL, Weng CH: Research and application of double-core buckling-restrained braces in Taiwan, *13WCEE* 2004:Paper No. 2179.

[13] Wada A, Nakashima M: From infancy to maturity of buckling restrained braces research, *Proceedings of 13WCEE*, 2004; (1732).

[14] Takeuchi T., Hajjar JF, Matsui R., Nishimoto K, and Aiken ID: Local buckling resistant condition for core plates in buckling restrained braces, *Journal of Constructional Steel Research*, 2010; 66(2):139–149.

[15] Lin PC, Tsai KC, Chang CA, Hsiao YY, Wu AC: Seismic design and testing of buckling-restrained braces with a thin profile, *Earthquake Engineering and Structural Dynamics*, 2016; 45(3):339–358.

[16] Wu AC, Lin PC, and Tsai KC: High-mode buckling responses of buckling-restrained brace core plates, *Earthquake Engineering and Structural Dynamics*, 2013; 43(3):375–393.

[17] Yoshida F, Okamoto Y, Murai M, Iwata M: A study on local failure of buckling restrained braces using steel mortar planks (Part-1: local failure), *Annual Meeting AIJ*, 2010; C-1, Structure III:961–962. *(in Japanese)*

[18] Takeuchi T, Hajjar JF, Matsui R, Nishimoto K, Aiken ID: Effect of local buckling core plate restraint in buckling restrained braces, *Engineering Structures*, 2012; 44:304–311.

[19] Tsai KC, Huang YC, Weng CS, Shirai T, Nakamura H: Experimental tests of large scale buckling restrained braces and frames. *Proceedings, 2nd Passive Control Symposium,* Tokyo Institute of Technology, 2002.

[20] Koetaka Y, Kinoshita T, Inoue K, Iitani K: Criteria of buckling-restrained braces to prevent out-of-plane buckling. *Proc. 14WCEE,* 2008.

[21] Hikino T, Okazaki T, Kajiwara K, Nakashima M: Out-of-Plane stability of buckling-restrained braces placed in chevron arrangement. *Journal of Structural Engineering, ASCE,* 2013; 139(11)

[22] Takeuchi T, Ozaki H, Matsui R, Sutcu F: Out-of-plane stability of buckling-restrained braces including moment transfer capacity. *Earthquake Engineering & Structural Dynamics*, 2014; 43(6):851–869,

[23] Takeuchi T, Matsui R, Mihara S: Out-of-plane stability assessment of buckling-restrained braces including connections with chevron configuration. *Earthquake Engineering & Structural Dynamics*, 2016; 45(12):1895–1917

[24] Nakamura H, Takeuchi T, Maeda Y, Nakata Y, Sasaki T, Iwata M, Wada A: Fatigue properties of practical-scale unbonded braces. *Nippon steel Technical Report*, 2000; (82):51–57.

[25] Matsui R, Takeuchi T: Cumulative deformation capacity of buckling restrained braces taking local buckling of core plates into account. *Proc. 15WCEE*, 2012.

[26] Takeuchi T, Ida M, Yamada S, Suzuki K: Estimation of cumulative deformation capacity of buckling restrained braces. *ASCE Journal of Structural Engineering*, 2008; 134(5):822–831.

[27] Wada A, Connor J, Kawai H, Iwata M, Watanabe A: Damage tolerant structure, *ATC-15-4, Proc. 5th US-Japan WS on the Imprement of Building Structural Design and Construction Practices,* 1992

[28] Kasai K, Fu Y, Watanabe A: Two types of passive control systems for seismic damage mitigation, *Journal of Structural Engineering*, *ASCE*, 1998

[29] Japan Society of Seismic Isolation: *Passive response control design manual, 2nd edition*, 2005 (*in Japanese and Chinese*)

[30] Takeuchi T, Yasuda K, Iwata M: studies on integrated building façade engineering with high-performance structural elements, *IABSE 2006*, 442–443

[31] Sutcu F, Takeuchi T, Matsui R: Seismic retrofit design method for RC buildings using buckling-restrained braces and steel frames. *Journal of Constructional Steel Research*, 2014; 101:304–313.

[32] Fujishita K, Sutcu F, Matsui R, Takeuchi T: Damage distribution based energy-dissipation retrofit method for multi-story RC building in Turkey, *Proc. IABSE*, 2015

[33] Qu Z, Kishiki S, Maida Y, Sakata H., Wada A: Seismic responses of reinforced concrete frames with buckling restrained braces in zigzag configuration. *Engineering Structures*, 2015; 105:12–21.

[34] Qu Z, Xie JJ, Wang T, Kishiki S: Cyclic loading test of double K-braced reinforced concrete frame subassemblies with buckling restrained braces. *Engineering Structures,* 2017; 139:1–14.

[35]Takeuchi T, Suzuki K: Performance-based design for truss-frame structures using energy dissipation devices, STESSA, 2003

[36] Takeuchi T, Xue SD, Nakazawa S, Kato S: Recent applications of response control techniques to metal spatial structures, *Journal of IASS*, 2012; 53(2), n.172:99–110.

[37] Takeuchi T: Retrofit of damaged gymnasia and towers according to response control concept, *Proceedings of 10CUEE*, 2013:17–24.

[38] Usami T, Lu Z, Ge H: A seismic upgrading method for steel arch bridges using buckling -restrained Braces, *Earthquake Engineering and Structural Dynamics*, 2005; 34(4-5, 10-25):471–496.

[39] Celik O, Bruneau M: Skewed slab-on-girder steel bridge superstructures with bidirectional -ductile end diaphragms, *ASCE Journal of Bridge Engineering*, 2011; 16(2):207–218

[40] Taga K, Koto M, Tokuda Y, Tsuruta J, Wada A: Hints on how to design passive control structure whose damper efficiency is enhanced, and practicality of this structure, *Proc. Passive Control Symposium 2004*, 105-112, Tokyo Tech, 2004.11

[41] Deierlein G, Ma X, Eatherton M, Hajjar J, Krawinkler H, Takeuchi T, Kasai K, Midorikawa M: Earthquake resilient steel braced frames with controlled rocking and energy dissipating fuses. *EUROSTEEL*, 2011

[42] Lai J, Mahin S: Strongback system: A way to reduce damage concentration in steel-braced frames, *Journal of Structural Engineering*, Vol. 141(9), 2014

[43] Takeuchi T, Chen X, Matsui R: Seismic performance of controlled spine frames with energy-dissipating members, *Journal of Constructional Steel Research*, Vol.115, 51–65, 2015.11

[44] Takeuchi T, Wada A: Buckling-restrained braces and applications, JSSI, 2017