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Review of Buckling-Restrained Brace Design and Application to Tall Buildings

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Abstract

Buckling-restrained braces (BRBs) are widely used as highly ductile seismic devices, with the first building using BRBs completed in 1989 in Tokyo, and thousands more now in Japan, USA, Taiwan, China, New Zealand and other countries. Although design codes of several countries specify BRB performance criteria, detailed design provisions are not necessarily provided, as BRBs are typically treated as a manufactured device. This paper briefly reviews the early history of BRB research and offers state-of-the-art views on the design criteria required to obtain stable and reliable performance. Representative project examples and up-to-date studies relevant to tall buildings are summarized.

Keywords: Buckling-restrained brace, Damage tolerant, Grid skin, Damped outrigger

1. Introduction

Buckling-restrained braces (BRBs) are seismic devices consisting of a primarily axially yielding core and an axially-decoupled restraining mechanism, which supresses overall buckling. As shown in Fig. 1, a typical restrainer consists of a steel hollow section filled with mortar, which encases a yielding core wrapped in a thin debonding layer.

The debonding layer or gap provided between the core and mortar (or all-steel restrainer) is an essential feature of modern BRBs, limiting axial load transfer to the restrainer by providing a low friction interface and accommodating lateral expansion of the core resulting from Poisson effects. As a result, the energy dissipation characteristics of BRBs are excellent and compare favourably to other fully ductile systems. For this reason, a properly designed BRB may be employed as a hysteretic damper, in many cases exhibiting sufficient fatigue capacity to safely withstand multiple design level earthquakes with no visible damage. Recently, a state-of-art textbook for the design and application of this device was published (Takeuchi and Wada, 2017). In this article, fundamental BRB design criteria and application concepts for tall buildings are discussed.

The basic concepts of buckling-restrained braces appeared in the 1970s, when limited experimental successes were reported by several researchers in Japan and India. The first practical BRB was achieved by Fujimoto et al., 1988 (Fig. 2(a)). 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 10- and 15-story steel frame office buildings in Tokyo (Fig. 2(b); Fujimoto et al., 1990). BRBs subsequently increased in popularity and other core and restrainer compositions soon followed, notably the all-steel tube-in-tube type.

Through the 1990s, BRBs were used in approximately 160 buildings in Japan. In 1992, the concept of a "damage tolerant structure" was proposed by Wada et al., 1992 and 1997, where BRBs are employed 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.

Collaboration with researchers and engineers in the US soon led to the first international application, with the construction of a building at UC Davis in 1998 and followed by an experiment at UC Berkeley in 2000 (Clark et al., 1999). Numerous other buildings with BRBs were soon constructed throughout California, including in seismic retrofit applications. In the early 2000s, buckling-restrained braced frames (BRBF) were first included in the Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-05). During these early years of technology transfer to international markets, a series of symposiums on passively controlled structures were held at Tokyo Institute of Technology, sharing code developments, BRB designs, and novel applications (Tokyo Institute of Technology,

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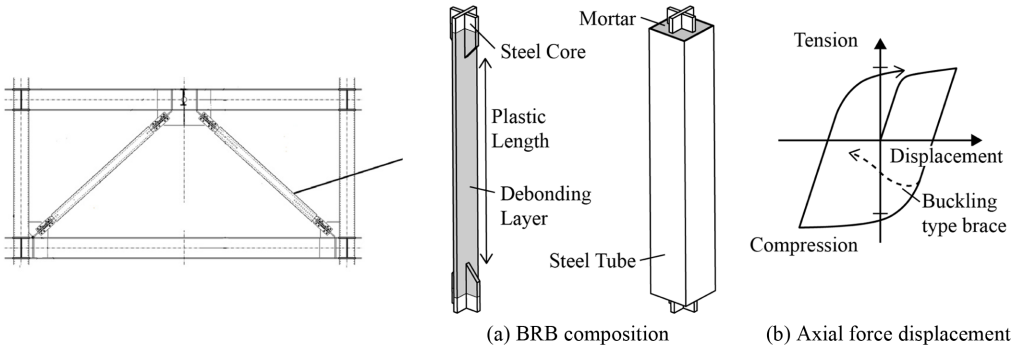


Figure 1. Concept of Buckling-restrained Brace.

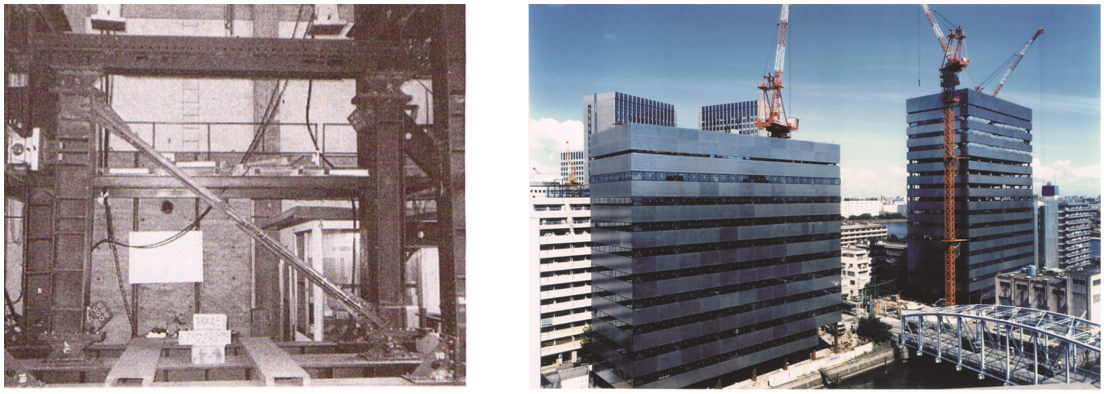


Figure 2. Early development of BRBs in Japan.

2000). Through the following decade, BRBs increased in popularity in numerous countries, from Taiwan in the early 2000s (Tsai et al., 2004) to the recent adoption in New Zealand as part of the Christchurch rebuild. BRBs are now widely known in seismic areas throughout the world and experimental research on BRBs may now be found in Japan, Taiwan, China, USA, Canada, Turkey, Iran, Italy, Romania, New Zealand, Chile and many other countries.

2. Requirements for Stable Hysteresis

Fundamentally, the BRB must be designed for strength and stability, considering both the local and global behavior of the core, connections, restrainer and adjacent frame, as shown in Fig. 3.

To obtain a stable hysteresis, the following design conditions must be satisfied (AIJ, 2009).

1. The *restrainer* successfully suppresses first-mode flexural buckling of the core
2. The *debonding layer* decouples axial demands and allows for Poisson expansion of the core
3. *Restrainer local bulging* owing to higher mode buckling is suppressed
4. *Global out-of-plane stability* is ensured, considering the potential for a hinge in the connections
5. The *low-cycle fatigue* and *peak displacement capacity* are sufficient for the expected demands

When designing the restrainer to suppress global buckling of the core, the restrainer flexural yield strength M_y^B should satisfy:

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

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, $N_{cu} = {}_d\alpha N_y$: core yield strength amplified by compression overstrength and strain hardening, and N_{cr}^E: Euler buckling load:

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

With EI_B the restrainer flexural stiffness, l_B the effective brace length and D_r the restrainer depth, and assuming initial imperfections $a_c/l_B \le 1/500$, a relatively slender restrainer $l_B/D_r > 20$, 330 MPa steel and and overall safety factor of ${}_e\alpha \ge 1.5$, Eq. (1) can be simplified to Eq. (3).

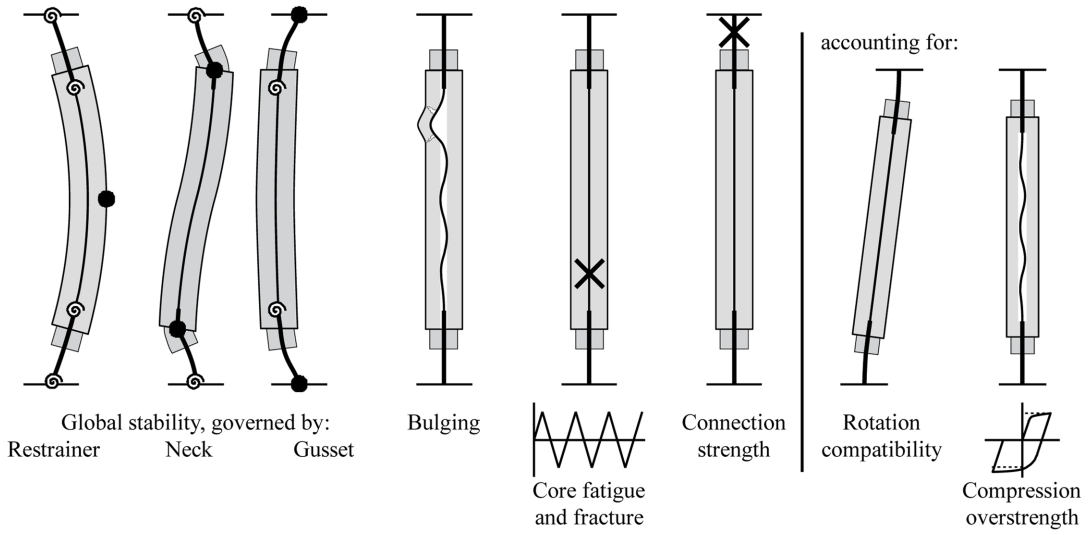


Figure 3. BRB strength and stability limit states.

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

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

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

where s_r: required clearance, ν_p: plastic Poisson ratio (=0.5), ε_{max}: maximum expected compressive strain (including strain amplification due to friction), and B_c: core width (or thickness).

3. Restrainer Local Bulging Failure

The compressible debonding layer or gap between the steel core and restrainer provides a space for the steel core to form high mode buckling waves when the BRB is under compression. An in-plane or out-of-plane local bulging failure may 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 for a rectangular core and RHS restrainer (Takeuchi et al., 2010; Lin et al., 2016; Takeuchi and Wada, 2017). Cruciform cores and/or CHS restrainers are less susceptible to bulging.

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

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

Eqs. (5) and (6) are validated against tests conducted in Taiwan and Japan in Fig. 4. The steel tube thickness (t_r), debonding gaps (s_{rs} and s_{rw}), loading sequence and mortar compressive strength all affect the bulging capacity. Mortar with insufficient strength may be crushed under the core normal forces, gradually becoming displaced and amplifying the bulging demand.

4. Global Instability Including Connections

To prevent a global instability initiated by yielding of the connections (Fig. 5(a)), two stability design concepts were proposed in the AIJ *Recommendations for Stability Design of Steel Structures*, 2009, and are shown in Figs. 5(b), (c). Note that the global inelastic buckling limit is generally governed by the neck or gusset when tested in a frame, as the restrainer axial loads are negligible.

(1) *Cantilever Connection Concept*: Effectively rotationally rigid adjacent framing and gussets are provided, so that the restrainer end continuity can be neglected. Stability is ensured by designing the connection as a cantilever, supported by the adjacent frame and gusset (Fig. 5(b)).

(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. 5(c)).

The *Cantilever Connection Concept* (Fig. 5(b)) relies on

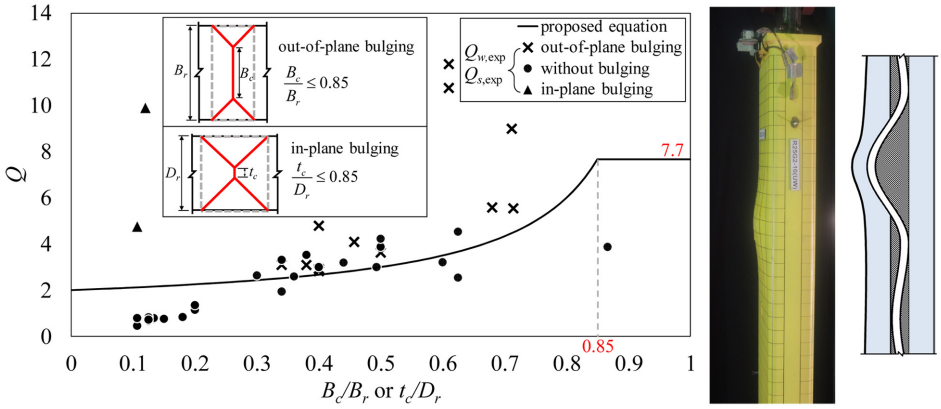
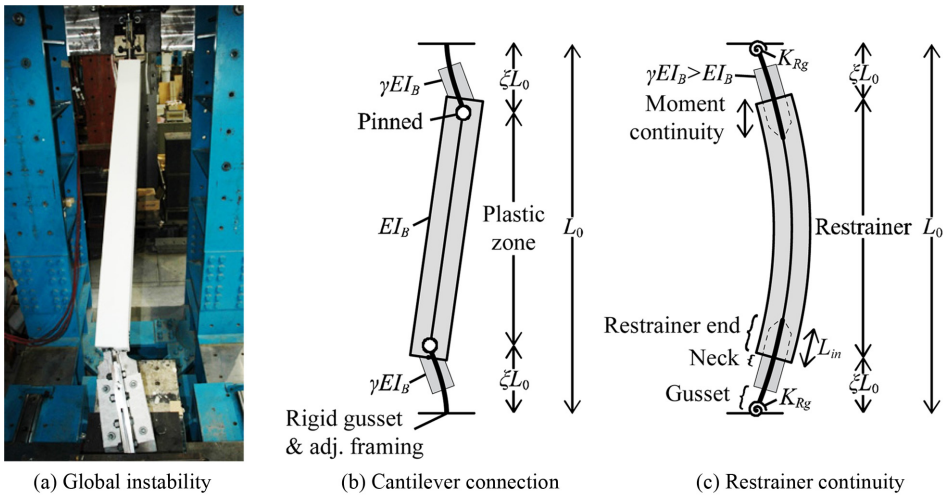


Figure 4. Comparisons between test results and proposed equations.



(a) Global instability (b) Cantilever connection (c) Restrainer continuity

Figure 5. BRB stability condition concepts (AIJ, 2009).

the gusset and adjacent framing rotational (or torsional) stiffness. The gusset rotational stiffness K_{Rg} is largely governed by the stiffener topology (Fig. 6), and this concept typically requires full-depth stiffeners corresponding to gusset types C or D. However, if a transverse beam and/or full-depth stiffeners are omitted (gussets type A or B), the connection stiffness rapidly decreases. This has a dramatic effect on the elastic buckling load, which can easily be less than 30% of the pure fixed-end cantilever buckling load. Thus, this stability concept is not suitable if unstiffened gussets are adopted.

The *Restrainer Continuity Concept* described in Fig. 5(c) is based on the analysis of the full BRB system with full flexural continuity provided at the restrainer ends. As BRBs are not monolithic, this is achieved via bearing action between the elastic core and restrainer along the insert length L_{in}. Both the neck and gusset must then be designed for the combined compression N_{cu} and buckling $P\delta$ demands. Although several design equations have been proposed, a generalized proposal by Takeuchi et al., 2014 has proven reasonably accurate for a diverse range of design situations.

$$N_{lim1} = \frac{(M_p^r - M_0^r)/a_r + N_{cr}^r}{(M_p^r - M_0^r)/(a_r N_{cr}^B) + 1} > N_{cu} \tag{7}$$

where N_{cr}^r is the inelastic buckling strength with pins at the restrainer ends, M_p^r is restrainer moment transfer capacity, M_0^r is imposed bending moment at restrainer end due to out-of-plane drift, 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:

$$N_{lim2} = \frac{[(1-2\xi)M_p^g + M_p^r - 2M_0^r]/a_r}{[(1-2\xi)M_p^g + M_p^r - 2M_0^r]/(a_r N_{cr}^B) + 1} > N_{cu} \tag{8}$$

where M_p^g is the plastic bending strength of the gusset plate reduced for the applied axial force, 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 exceed N_{cu}.

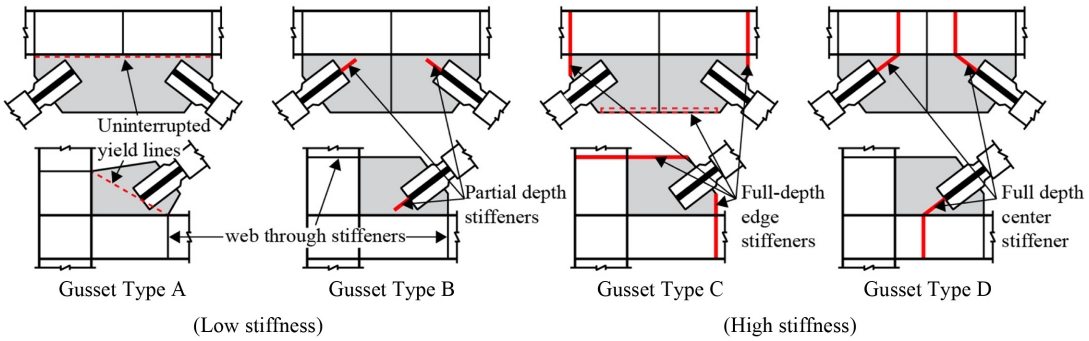


Figure 6. Gusset plate types and out-of-plane stiffness.

5. Cumulative Deformation Capacity until Fracture

The cumulative deformation capacity of a BRB subjected to constant-amplitude axial displacements can be roughly modeled following Manson-Coffin's rule. The fatigue performance of BRBs is reduced compared to the underlying steel material due to the bending strains and non-uniform axial strain distribution in the core plates caused by higher mode plastic buckling within the debonding gap and friction (Fig. 7). Therefore, it should be noted that the low-cycle fatigue is sensitive to the debonding gap design and fabrication tolerances (Matsui et al., 2012). The fracture criteria under a random amplitude response may be evaluated from the nominal axial strain history (axial displacement divided by core yielding length) using Miner's rule and supplier-specific low cycle fatigue curves. Alternatively, Takeuchi et al. proposed the criteria given by Eq. (9), which uses averaged amplitudes and does not require detailed strain time-histories (Takeuchi et al., 2008).

$$\chi = \frac{1}{\dfrac{\alpha_s}{\chi_{so}} + \dfrac{(1-\alpha_s)}{4}\left\{\dfrac{\overline{\Delta\varepsilon_{ph}}^{(1+m_2)}}{C}\right\}^{\frac{1}{m_2}}} \tag{9}$$

where $\overline{\Delta\varepsilon_{ph}}$ = half of the average plastic strain amplitude. Eq. (9) gives the same criteria as the Miner's rule when the exponential value of the fatigue curve m_2=1 (Matsui et al., 2012).

6. Damage Tolerant Structures

Over the past 30 years, a variety of design concepts using BRBs have been developed and realized. In 1992, Wada et al. proposed the concept of a "damage tolerant structure," where the main frame remains elastic and energy dissipation devices are placed in parallel (Fig. 8, Wada et al., 1992). An early example is the Triton Square Project, a 40-story (180 m) office building located in Tokyo (Fig. 9). The frame employs high strength HT780 columns and HT590 beams, and LY100 BRBs. While the BRB layout introduces some inefficiencies owing to an indirect brace configuration, the low yield strength of LY100 ensures a small yield drift angle. In the late 1990's, optimal distribution methods of BRBs using equivalent linearization techniques were developed and applied in these damage tolerant designs (Kasai et al. 1998).

Fig. 10 shows a 24-story (133 m) damage tolerant structure completed in 2001 in Fukushima, Japan. BRBs in the lower stories control the seismic response, while viscoelastic dampers at middle stories are effective in both seismic and wind vibration. This building was subjected to design level ground motions during the 2011 Tohoku Earthquake and achieved immediate occupancy.

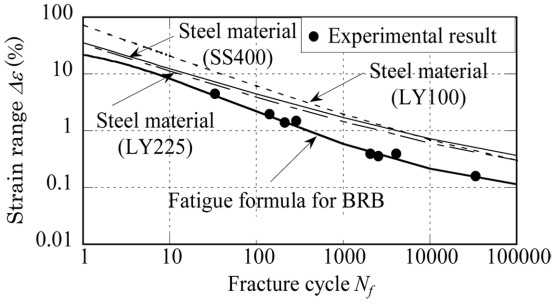


Figure 7. Low-cycle fatigue capacity example for BRB and steel material.

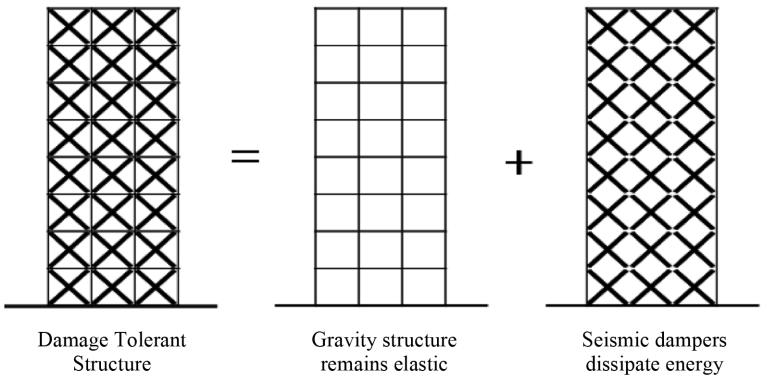


Figure 8. Concept of damage tolerant structure.



Figure 9. Triton Square, 1992.

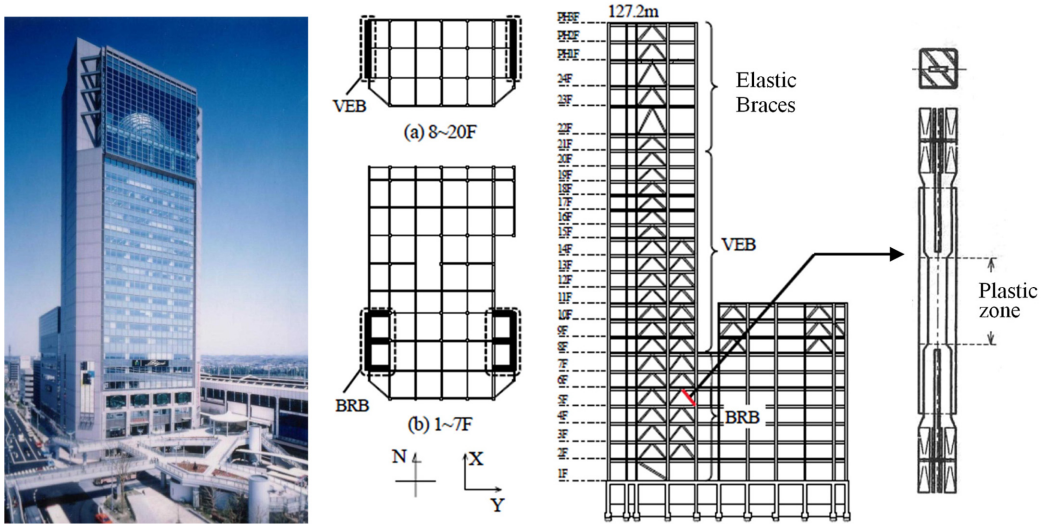


Figure 10. Koriyama Big-Eye Building.

A BRB yield story drift of just 0.13-0.16% was achieved by using a low yield steel (LY225) and short plastic length (L_p/L_0 = 0.25-0.3). In the 2011 Tohoku Earthquake, the main frame remained elastic, while deformation recording devices indicated a workpoint displacement ductility of $\mu \approx 3.8$ and cumulative plastic strain of $\sum\varepsilon_p \approx 22\%$ ($\sum\varepsilon_p/\varepsilon_y \approx 200$) in some BRBs. This damage was less than 6% of the low cycle fatigue limit, leaving plenty of residual capacity for expected future strong ground motions and justifying the decision to leave all BRBs in place.

Another example of a damage tolerant structure is the Grand Tokyo (Fig. 11), a 205 m high, 43-story building in Tokyo completed in 2007. This building is a typical example of BRB application in high-rises built in major cities in Japan. Most of the recent 200 m-class buildings are constructed using passive control devices, with a large proportion adopting BRBs each year. BRBs with yield strength of 10000 kN or greater (Fig. 12) are typically used for these structures.

7. Grid-skin Structures Using BRBs

A "grid-skin structure" can be defined as a structural system with vertical and diagonal perimeter members forming the primary seismic and wind resisting frame, encompassing the concepts of a "braced tube" or "diagrid system" (Takeuchi, 2015; Fig. 13). This enables large interior spaces free of seismic braces or walls. Recently, several applications have been applied to tall buildings using BRBs as energy-dissipating members. The perimeter frames enveloping the building not only support the vertical weight, but also provide horizontal stiffness and strength against seismic and wind loads. As a result, no stability elements such as diagonal braces or concrete shear walls are required in internal spaces, which permits a flexible interior design and renovation throughout the building's lifetime. A recent example is shown in Fig. 14. This 205-m high building completed in San Francisco is covered by damped mega-braces composed of BRBs and viscous dampers, producing equivalent damping of 8%

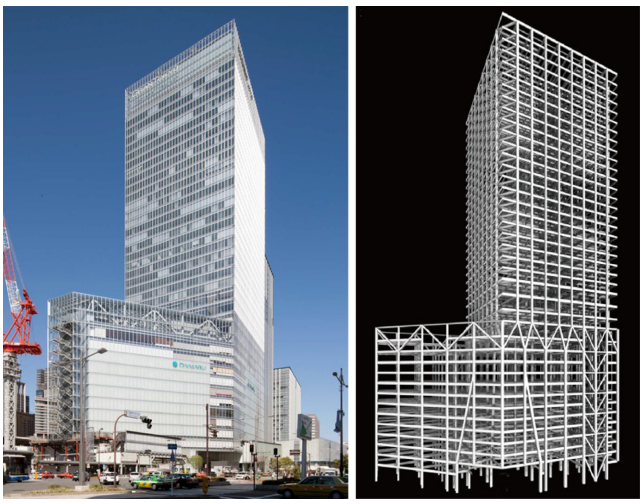


Figure 11. Tokyo BRBF (Gran Tokyo North).



Figure 12. Erection of large BRBF.

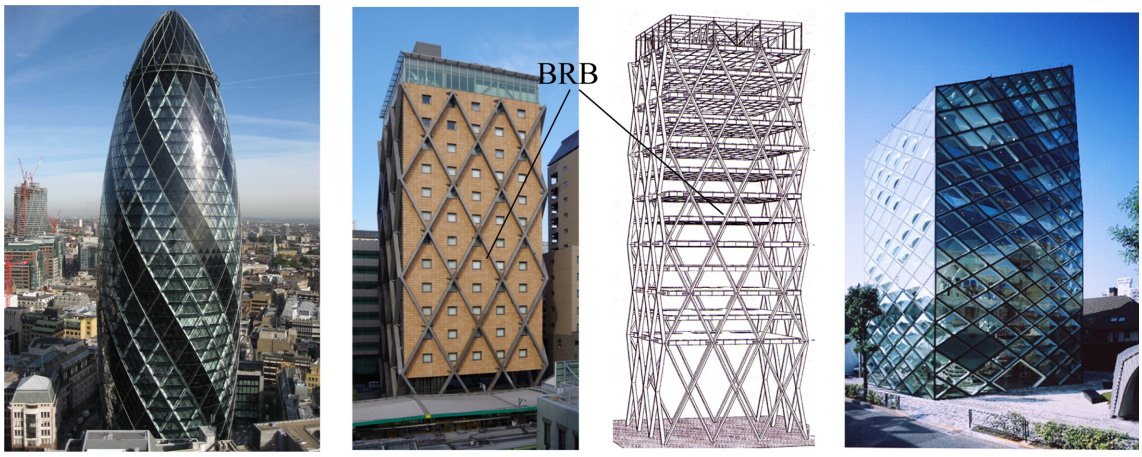


Figure 13. Grid-skin structures.



Figure 14. 181 Fremont Tower, San Francisco (Alumfti et al., 2018).

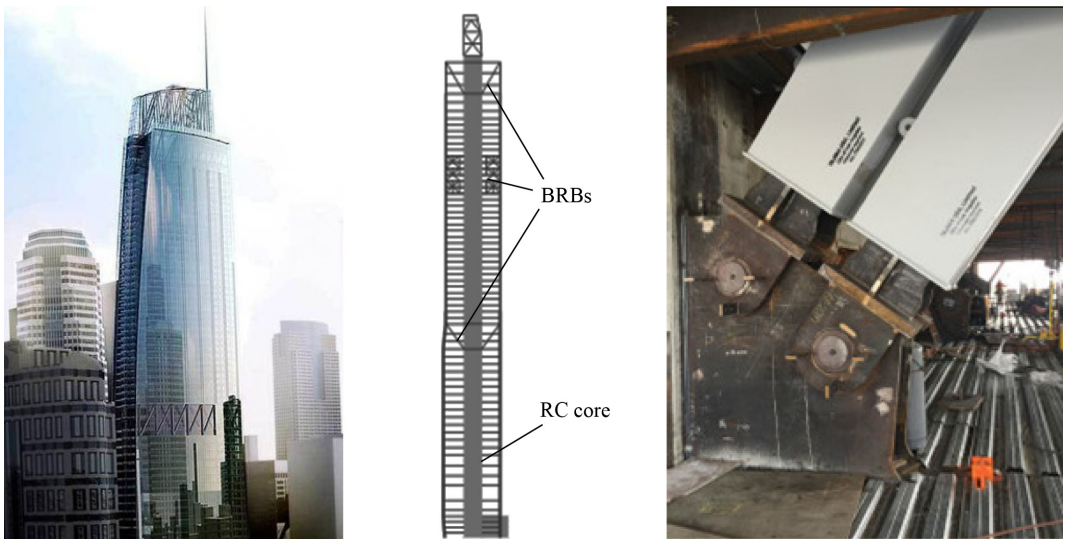


Figure 15. Wilshire Grand Tower, LA (Smith et al., 2007).

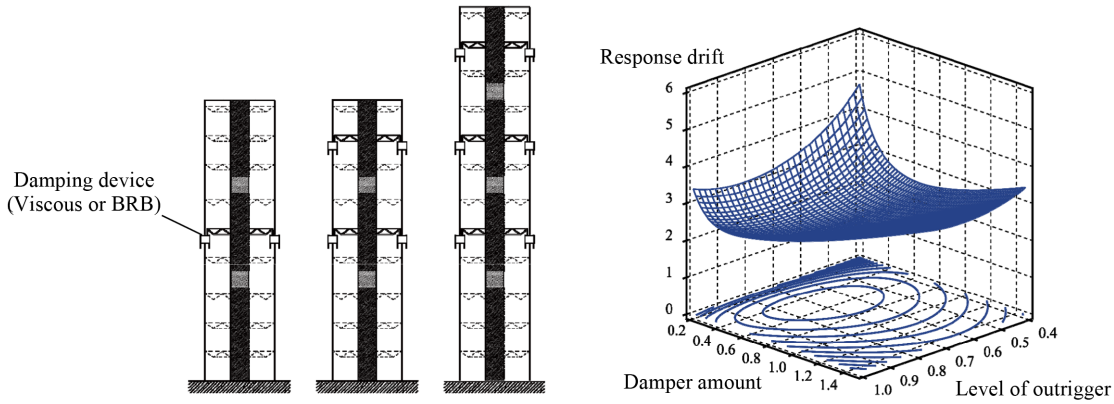


Figure 16. Damped outrigger and optimal design (Huang et al., 2017).

and achieving immediate re-occupancy with limited disruption after a 475-year earthquake (Alumfti et al., 2016).

8. Damped Outrigger Using BRBs

The damped outrigger concept has also become increasingly common to control wind and/or seismic demands in tall buildings (Smith et al., 2007). Fig. 15 shows the Wilshire Grand Tower, a 335 m, 73-story building in Los Angeles. BRBs are used as outrigger dampers, with up to a 10000 kN capacity achieved by installing 4 BRBs in parallel (Joseph et al., 2017). It is known that there is optimal outrigger level and optimal amount of damper for maximizing the damping effects and minimizing the response as shown in Fig. 16 (Huang et al., 2017). The same characteristics are also reported for damped outriggers using BRBs (Lin et al., 2018).

Another promising system that has seen recent practical application introduces BRBs in parallel with an elastic spine (Taga et al., 2004; Lai et al., 2014) or at the base of a rocking elastic spine frame (Deierlein, 2011; Chen, 2018; Simpson, 2018), preventing damage concentration at weak stories.

9. Conclusions

This paper briefly introduced the early history and key design criteria for BRBs, followed by representative project examples and design concepts for tall buildings. These concepts enable enhanced resiliency and seismic performance, and further investigations are expected in the future.

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