DOI: https://doi.org/10.14256/JCE.3778.2023 Građevinar 12/2023

Primljen / Received: 21.4.2023.
Ispravljen / Corrected: 14.8.2023.
Prihvaćen / Accepted: 20.10.2023.
Dostupno online / Available online: 10.1.2024.

A critical review on buckling restrained braces

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Research Paper

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A critical review on buckling restrained braces

Currently, buckling restrained braces are gaining popularity in earthquake resistant designs. These braces facilitate stable hysteretic behaviour with a non-buckling steel core encased in a steel tube, which is filled with concrete or mortar. However, in the last few years, researchers have observed that there is no need for the filler material and these braces can be made of steel. This paper presents a summary of buckling restrained braces on the grounds of numerical and experimental research results and attempts to summarize the basic design provisions according to the American standards and recommendations from the available research. The paper also discusses the concept, stability criteria, initial and post stiffness, energy dissipation capacity, failure modes observed, and the practical applications of buckling restrained braces, followed by conclusions and future suggestions for the development of these braces.

Key words:

structures, steel, braces, buckling, restrainer, core, energy dissipation

Prethodno priopćenje

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Kritički osvrt na vezove sa spriječenim izvijanjem dijagonala

Vezovi sa spriječenim izvijanjem dijagonala postaju sve popularniji u konstrukcijama otpornima na potrese. Takvi vezovi olakšavaju stabilno histerezno ponašanje dijagonala s čeličnom jezgrom koja se ne izvija te je obložena čeličnom cijevi ispunjenom betonom ili mortom. Međutim, u posljednjih nekoliko godina istraživači su primijetili da nema potrebe za ispunom te da vezovi mogu biti izrađeni od čelika. U ovom je radu prikazan sažetak svojstava vezova sa spriječenim izvijanjem dijagonala na temelju numeričkih i eksperimentalnih rezultata istraživanja te se prema američkim normama i preporukama iz dostupnih istraživanja sažimaju osnove za projektiranje. U radu se također raspravlja o konceptu, kriterijima stabilnosti, početnoj i naknadnoj krutosti, sposobnosti rasipanja energije, uočenim načinima sloma te praktičnoj primjeni vezova sa spriječenim izvijanjem, nakon čega slijede zaključci i prijedlozi za budući razvoj tih vezova.

Ključne riječi:

konstrukcije, čelik, vezovi, izvijanje, sustav koji sprječava izvijanje, jezgra, rasipanje energije

1. Introduction

Frames equipped with buckling restrained braces (BRBs) have been extensively applied in current years for wind and earthquake load resistant structures. Such frames offer almost equal axial yield forces in tension and compression and therefore yield without significant buckling. Owing to this, they offer a better alternative to conventional frames. In other words, the buckling restrained braced frame is a type of conventional braced frame that prevents the buckling of the brace under compression. Figure 1 shows the comparison of the hysteretic behaviour of a BRB with the conventional brace system. The concept of a BRB is simple; particularly, it restrains the buckling of the brace before yielding. The brace is made by encasing a steel core inside a restraining arrangement. The space between the outside restrainer and the steel core is filled with a filler material, which may be concrete or mortar. This restrains the inner steel core against buckling. The core is coated with an unbonding material before adding the filler material to avoid adhesion between the core steel and filler. This also prevents the transfer of the axial load between the core and restrainer. The idea of restraining the core leads to uniform and symmetrical hysteretic behaviour in BRBs and prevents the buckling of the brace. Figure 2 shows the concept of the BRB [1].

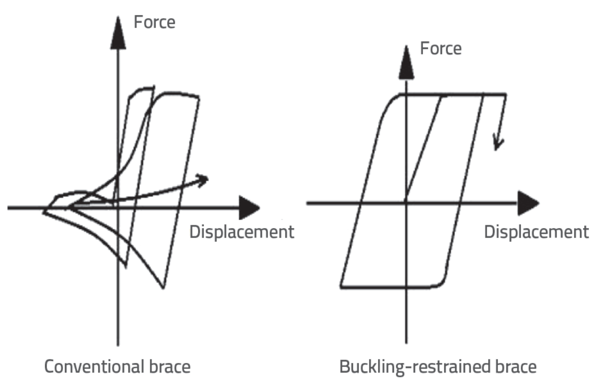

Figure 1. Hysteretic behavior of a conventional brace compared to a BRB

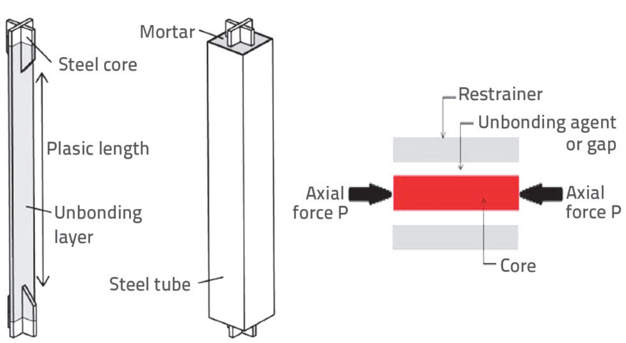

Figure 2. Concept of the BRB

The seismic stability of any structure can be achieved through various methods, either by inducing dynamic oscillators and energy dissipating devices or by developing a seismic control system within the structure. Seismic control systems can be active, passive, or hybrid. The base isolation of the structure is another similar system. The use of any seismic control systems is costly and is required in areas that are highly seismic-prone. Control devices such as yielding metallic devices, friction devices, fluid viscous devices and viscoelastic devices are preferred in bridge construction for seismic safety. To ensure seismic safety in medium earthquake-prone areas, energy dissipating devices are preferred owing to their low cost and ease of installation. Prior to the development of BRB frames, eccentrically braced frames (EBFs) were being considered as better alternatives to moment resisting frames owing to their higher elastic stiffness and better energy dissipation capacity. A capacity design approach has also been studied for the design of the size of braces in EBFs [2]. The ductile shear links in EBFs are efficient seismic energy dissipating devices that remain elastic on linear loading and resist non-linear plastic deformation on any seismic loading. Seismic links have been included in the design provisions of various countries. The design procedure for shear links as per loading conditions, web-stiffener specifications, and connection with columns have been studied by researchers [3]; however, the performance of links on unequal link end moments and axial forces still need further research. Although web-buckling remained the limitation of shear links, attempts have been made to improve their buckling behaviour by adding a stiffener to the web. Short seismic links with web stiffeners designed as per Eurocode 8 have been tested in the past and have demonstrated adequate reliability for the reliability class RC2 with 50 years of the mean recurrence interval [4]. In fact, the design provisions of BRB frames were developed from those of EBFs.

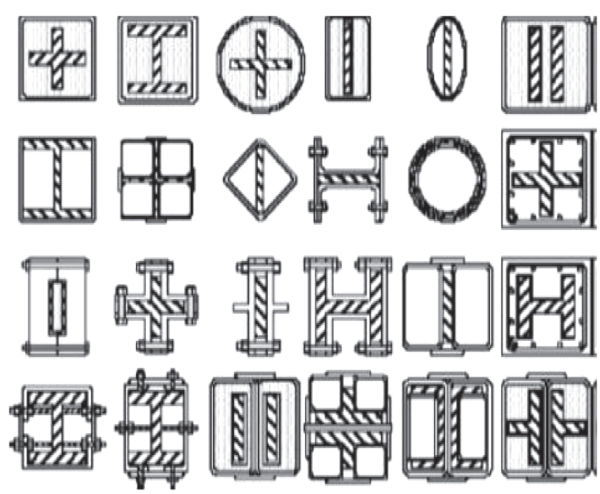

Figure 3. Different cross-sections of BRB, [5]

The cross-section used for the core section of the BRB is generally rectangular shaped; however, various other shapes can be used. Figure 3 shows the various cross-sections of BRBs adopted by researchers in the past few decades [5]. The base design of the BRB makes it heavily weighted and difficult to handle. Curing the filler material, which may be mortar or concrete, is also very difficult in these braces. To eliminate these difficulties, all-steel BRBs were introduced. The idea was to remove the filler material from BRBs, to make them lighter and easier to cast. In all-steel BRBs, the inner core is encased in an outer tube with no filler material in between. The use of unbonding material is also not mandatory. Instead, a gap is provided between the restrainer and core member to provide free movement of the core inside the restrainer casing. However, the working principle is similar to that of conventional BRBs. All-steel BRBs are easier to fabricate, economical, and provide an ease of inspection after an earthquake because they can be disassembled. In this paper, the authors present a critical summary of BRBs on the grounds of their concept, development, stability criteria, experimental and numerical research, design considerations, failure modes to be observed, and applications to date. A detailed study and conclusion made on BRBs is presented. In today's world, where the damage from earthquakes is becoming unpredictable, the study of such an economical and efficient energy dissipating device is worthwhile. The authors also attempt to propose some suggestions on the basis of the available literature. The authors believe that this paper will be helpful to other researchers in the study of BRBs and will bridge the gap in research so that further studies in this field can be made.

2. Development of BRB

2.1. Concept of BRB

A BRB is composed of the following components:

1. **Buckling restraining mechanism (BRM):** This is the outer cover and the axial force carrying unit. The BRM used earlier was a mortar-filled steel unit and was made exceptionally rigid. However, the new generation BRM system is comparatively lighter as it is made of steel.
2. **Core:** It is the main buckling restraining unit whose function is to prevent the brace from buckling. It is made of high strength steel and any cross-section shape. Generally, the core is preferred to be rectangular along the yield length and cruciform along the end length portion. It can also be circular or any other shape according to the designer's interest.
3. **Unbonding material:** The unbonding materials are employed in between the core plate and restraining members to provide space for the expansion of the core plate under compression. They reduce the adhesion force between the core plate and restraining members. Various unbonding materials have been used by researchers. A layer of epoxy resin and silicon resin were considered as the preferable unbonding materials. Vinyl tapes, high density styrofoam sheets, chloroprene rubber, rubber sheets, and silicone sheets have also been used for this purpose. Unbonding materials should be employed as separation units, otherwise, a gap should be maintained between the two units. Unbonding materials are not essential in most all-steel BRBs. However, for high performance BRBs with a long yielding segment and thin core plate, the unbonding materials are still necessary to obtain higher low-cycle fatigue properties. The main function of the unbonding material is not only to reduce the adhesion force between the core plate and restraining members but also to provide space for the expansion of the core plate under compression.

2.2. History and development

The idea of BRB originated in Japan. Wakabayashi et al. [6] were the first to experiment on these braces by introducing panel BRBs. Kano et al. [7] conducted a numerical study on the elasto-plastic behaviour of BRBs. Kimura et al. [8] proposed the concept of BRBs to fabricate a brace that addressed the degradation problem of the bearing capacity and stiffness. The proposed BRB also helped to meet the requirements of the reduction of the ductility and energy dissipation capacity of the ordinary steel brace owing to compression. Mochizuki et al. [9, 10] carried out research to address the problem of the overall stability of the steel brace surrounded by reinforced concrete. The early Japanese researchers of BRB also proposed that the steel core should be coated with an unbonding agent to de-bond the core from the surrounding concrete. Apart from the research studies that were conducted on these braces with a good response, BRBs were not included for design recommendations in the Architectural Institute of Japan (AIJ) before 1996. Fujimoto et al. [11] conducted research on BRBs with a steel core encased in steel tubes filled with concrete or mortar. These BRBs were first applied in two steel framed office buildings in 1989 and thereafter, were employed in 160 buildings in Japan [12]. By 1990, hundreds of buildings in Japan accepted and applied BRBs, most of which were taller than 15 stories. Moreover, Wada et al. [13] proposed a new 'damage tolerant' design concept. In this concept, they introduced BRBs as energy dissipating elasto-plastic dampers within an elastic main frame. The acceptance and application of BRBs in Japan increased mainly after the 1995 Kobe earthquake.

Although the research originally started in Japan, the technology was transferred to the United States of America (USA) after the good response of these braces in Japan. The Northridge earthquake in 1994 made a significant change in the steel seismic research in the United States of America. Prior to this earthquake, it was believed that special moment resisting frames were effective solutions to the earthquake resistant design of steel structures. However, the brittle failure of the beam-to-column moment connections that occurred in many multi-storey steel buildings, forced researchers to rethink and reconstruct the seismic design provisions for the earthquake resistant design of structures. The first practical application of BRBs in the US was in the construction of a building at UC Davis in 1998 whose testing took place in 2000 at the University of California (UC) Berkeley. In a couple of years, several projects using BRB were executed. Black et al. [14] conducted component testing on the braces and observed a repeated symmetrical hysteretic behaviour. The seismic behaviour of these braces was widely investigated by Sabelli et al. [15]. Several buildings were then constructed in the US with BRBs installed in them, after the inclusion of the design guidelines for BRB frames in the seismic provisions for structural steel buildings [16]. The pseudo-dynamic numerical analyses of the braces were investigated by Fahenstock et al. [17] on a large scale. AISC 341-10 (Seismic Provisions for Steel Structures) provides the design standards for these braces [18].

Many research studies have been conducted on the seismic resistance of structures. Strengthening existing buildings and the buckling resistance of structural components have been a topic of interest among researchers. Numerical studies on the buckling resistance of pin-jointed stainless-steel columns having angled sections designed as per Eurocode 3 [19] have been conducted [20]. Investigations have also been made on different-shaped buttresses for application as supporting structures against seismic effects [21]. Considerable resistance to seismic effects can be achieved by adding light-weighted timber storeys to an existing structure [22]. The change in stiffness of the structure is due to the added timber storeys. Outrigger structures for the seismic safety of tall buildings can also be a good alternative [23]. Performance-based design for strengthening existing buildings is the research direction for upcoming researchers. A case study on strengthening an existing mid-rise building was conducted by Erdem et al. [24]; it involved comparing the seismic performance of the building as per the Turkish code and American standards through non-linear analysis. They then proposed design methods based on the performance of the building.

Among Asian countries, China has been an active country as far as the application and development of energy dissipating devices is concerned. Similar to Japan, China is also an earthquake prone country and has therefore expressed a high interest in BRBs. Majority of the research conducted on these braces in the last two decades is from China. The design consideration for energy dissipating devices was included in the Chinese codes of the seismic design of buildings in the early 2000s only [25]. Xie [26] evaluated the practical application of BRBs in Asian constructions. Subsequently, research was conducted on BRBs in Asian countries other than Japan, especially in China. The technical specifications for steel structures in tall buildings [27] were issued by China and included the design criteria for BRBs. The technical specifications for the application of BRBs [28] were issued by the China Association for Engineering Construction Standardization for the design provisions of these braces along with gusset plate design guidelines for the connection of the braces.

Several other countries across the world have shown interest in BRBs by conducting experimental and numerical research over these braces. Countries such as Canada, Turkey, New Zealand, Iran, Taiwan, South Korea, Europe, and India have also made their contribution in the research and development of BRBs by proposing various modifications based on their findings.

3. Stability analysis of BRBs

There are three major buckling modes under which BRBs are identified:

- Global buckling of the brace under axial compression.
- Local buckling of the metallic core.
- Torsional buckling of a portion of the extended part of the core outside the outer tube.

3.1. Global buckling prevention criteria in BRBs

The global stability of the brace is the most important criteria and must be studied to identify the behavior of BRBs. Figure 4 shows the mechanism of global buckling in BRBs.

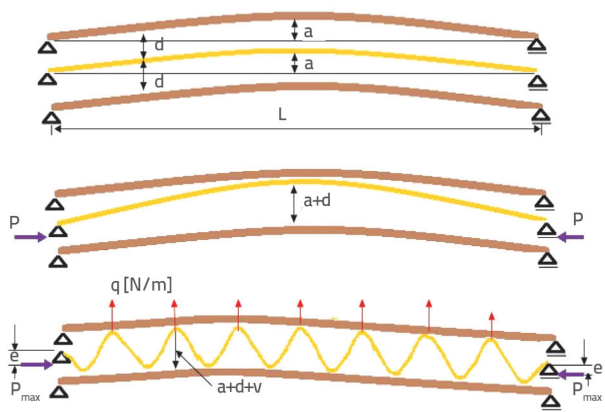

Figure 4. Performance of BRB on global buckling

Currently, the most popular relation to prevent overall buckling in BRBs simply supported at both ends was given by JSSC [29], equation (1)

$$\frac{P_{max}(a+d+e)}{1-\dfrac{P_{max}}{P_E^R}} \le M_y^R \tag{1}$$

where M_y^R is the yield moment of the restraining unit, P_{max} is the maximum axial compressive force experienced by the brace, P_E^R is the Euler buckling load induced in the restrainer, a is the initial deformation of the brace at the center, d is the space between the core member and resisting member, and e is the eccentricity of the axial compressive force, which is same at both the ends.

The left side of the equation gives the bending moment at the center of the core member taking the P-delta effect into consideration, whereas the right side of the equation shows the yield strength of the core member when buckling. It also shows that the yielding at initial stage is assumed to be in the limit state. After the overall buckling, the bending moment of the core member at the center is expressed as in equation (2) [30]:

$$M_c = P_{max}(a + d + e + \nu) \tag{2}$$

where ν is the lateral deformation in the restrainer after overall buckling and is given by equation (3),

$$\nu = 5M_cL^2/48E_RI^2 \tag{3}$$

where L is the length of the brace, E is the elastic modulus, and I is the second moment of inertia of the outer tube.
Substituting the above value in equation (2), we obtain equations (4) and (5)

$$M_c = P_{max}(a + d + e + 5M_cL^2/48E_RI^2) \tag{4}$$

$$M_c \cong \frac{P_{max}(a+d+e)}{1-1.03\frac{P_{max}}{P_E^R}} \tag{5}$$

where the Euler buckling load of the restrainer is given by equation (6)

$$P_E^R = \frac{\pi^2 E^R I^R}{L^2} \tag{6}$$

Hence, it can be observed that when $P_{max} = P_y$, the ratio of the buckling load on the restrainer, P_e, to the yield buckling load of the core member, P_y, plays a significant role in the evaluation of the bending moment. Thereafter, equation (7) was proposed including both P_e and P_y [31].

$$\frac{\varphi P_e}{1.3P_y} \geq 1 \tag{7}$$

where φ is the strength reduction factor whose value was 0.85 and the above equation was equation (8)

$$\frac{P_e}{P_y} \geq 1{,}5 \tag{8}$$

Fujimoto et al. [11] in his analysis, proposed that the critical compressive load for the brace, in which a steel core member is encased by a resisting unit, can be obtained from the solution of the equilibrium in equation (9),

$$E_bI_b\frac{d^2v}{dx_2} + (\nu+\nu_0)N_{max} = 0 \tag{9}$$

where E_bI_b is the flexural stiffness of the restraining member, N_{max} is the yielding load of the BRB, ν is the transverse deflection, and ν_0 is the initial deflection of the core. It is assumed that the initial deflection, ν_0 is a sinusoidal curve, equation (10),

$$\nu_0 = a \sin px/L \tag{10}$$

The solution to the equilibrium equation yields equation (11)

$$\nu + \nu_0 = \frac{a}{1-\frac{P_{max}}{P_e}} \tag{11}$$

where N is replaced with P. The bending moment at the center of the brace is expressed in equation (12)

$$M_c = \frac{(a+d+S_r)}{1+\frac{P_{max}}{P_e}}P_{max} \tag{12}$$

The strength and stiffness required by the restraining unit can be obtained by assuming that the buckling of the brace takes place when the stress in the outer tube is equal to the yield stress of the core i.e., the maximum axial force in the brace reaches the yield load experienced by the steel core, equation (13).

$$\frac{P_e}{P_y} \geq 1 + \frac{\pi^2 E_b(a+e+S_r)D}{2\sigma_y L_b^2} \tag{13}$$

where D is the depth of the restraining member, σ_y is its yield strength, and L_b is its length. The influence of the gap amplitude has not been included in the above equation when determining the moment. Hence, by including that, the equation of the bending moment at the centre becomes

$$\frac{P_e}{P_y} \geq 1 + \frac{E_b(a+e+S_r)D}{2\sigma_y L_b^2} = \beta \tag{14}$$

Equation (14) suggests that the overall buckling stability of the brace is ensured if the ratio of the buckling load on the restrainer P_e to the yield buckling load of the core member P_y is not less than β. This term β, governs the global stability criteria of BRBs and depends on the material and geometric behaviour of the brace.

3.2. Local buckling of the metallic core

For the local buckling mode, the efficiency of the BRBs can be improved when the buckling of the inner core along the restrained length does not occur. Wada et.al [13] gave an equation (15) for the critical load for the local buckling of the core,

$$P_{cr} = 2\sqrt{\beta E_i I_i} \tag{15}$$

where E_iI_i flexural rigidity for the inner steel core and β distributed spring constant.
The high order buckling of the inner steel core can be avoided when,

$$P_{cr} \geq \sigma_y A_i \tag{16}$$

this requires

$$\beta \geq \frac{\sigma_y^2 A_i^2}{4E_i I_i} \quad (17)$$

where A_i Cross section area of the inner core.
It was observed that in higher modes, the critical load of the inner steel core does not depend on the end conditions of the core.

3.3. Torsional buckling in the portion of the extended part of the core outside the outer tube

The portion of the core that extends from the casing may undergo torsional buckling, which is the third and most critical mode of buckling for BRBs. Many studies have been conducted and some are still in progress on the torsional buckling behaviour of the unbonded braces. It was observed that the critical load causing torsional buckling of the extruded part in BRBs does not depend on the length of the extension.

4. Energy dissipation capacity of BRBs

4.1. Hysteretic behaviour of BRBs

A conventional braced frame is expected to resist the distortions of the frame owing to lateral forces produced in any seismic event. Whenever any seismic hazard occurs, the braces induced in the frame are subjected to repeated stress cycles. Because conventional steel is weak during compression, it buckles and exhibits unsymmetrical hysteretic behaviour. Hence, the ability of the brace to dissipate energy shrinks. Moreover, the buckling behaviour of a conventional brace is highly unpredictable. This failure mode of the braced frame can be counterattacked by BRB frames. BRBs resist both compression and tension forces when their elastic limits are exceeded, dissipating energy while still maintaining the structure. When subjected to repeated cyclic loading, a BRB frame system exhibits a symmetrical and stable hysteretic loop in both tension and compression, as illustrated in Figure 5.

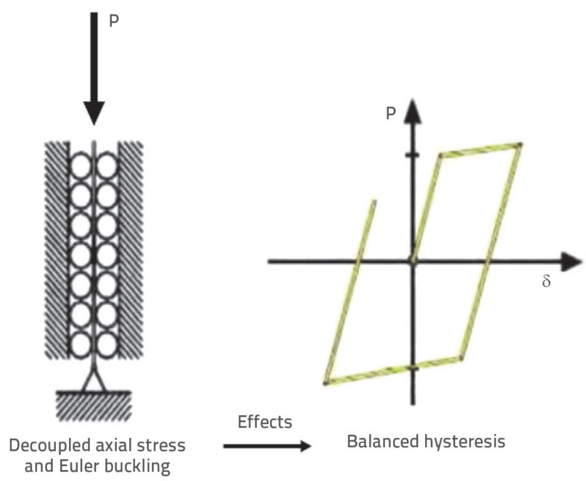

Figure 5. Behaviour of BRB [32]

The most crucial quality of the BRBs is this trait. A BRB system is regarded as superior to a traditional braced frame system solely because of this trait. Less loads and deformations in the brace are redistributed owing to the stable hysteretic nature. The ability of BRBs to dissipate energy decreases the overall structural damage. Because these braces do not buckle laterally, there is less damage to the adjacent non-structural units [15].

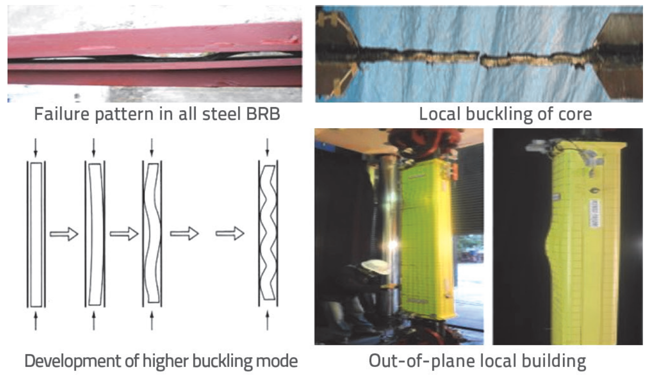

Figure 6. Failure modes in BRB [1, 33]

4.2. Cumulative deformation capacity of BRB

One of the important criteria that governs the energy dissipating behaviour of BRBs is its cumulative deformation capacity up to the core fracture. The local buckling of the core member results in non-uniform strain distribution along its length. This phenomenon decreases the cumulative deformation capacity in the brace. Takeuchi et al. [34] from previous research proposed a formula for the normalized deformation amplitude $\Delta\varepsilon_n$ for the low-cycle fatigue of BRBs,

$$\Delta\varepsilon_n = 0{,}5\, N_f^{-0.14} + 54\, N_f^{-0.71} \quad (18)$$

Where N_f is number of fracture cycles.
Equation (18) was then modified by substituting the results by Takeuchi et al. [35] to obtain

$$N_f = \left(1 - \frac{3\Delta\varepsilon_n}{70}\right)\left(\frac{\Delta\varepsilon_n^{-1.41}}{3.63 \cdot 10^{-3}}\right) \quad (19)$$

Where $N_f > 20$.

Equation (19) defines the partial concentration of the plastic strain in the core member at ultra-low fatigue failure zones. It indicates a condition with stress exceeding the limiting value with a negative tangent modulus. Takeuchi et al. [35] defined a ratio to calculate the local plastic strain without considerable effort. This ratio is obtained by dividing the local strain at the plastic stress concentration point with normalized deformation. The ratio was initially proposed for braces with circular tubes. The point where the local strain is approximately equal to the value calculated by the fatigue formula, is regarded as the fracture point of the brace.

5. Research on BRB

To date, several studies have been conducted on BRBs in different ways, including the components of BRBs, sub assemblage of BRBs or the entire structure installed with BRBs. Research conducted on BRBs can be broadly divided into two parts: experimental and numerical.

5.1. Experimental studies on BRBs

The experimental research on BRBs began in the early 1990s followed by a number of investigations in the next two decades.
A generalized summary of the experimental research made to date is presented in Table 1.

Table 1. Summary of experimental studies on BRBs

Reference \ Parameters	n	b	t	L	Core material	Investigated parameter
Fujimoto et al. 1990 [12]	5	190	19	3190	rolled steel JIS G 3101 SS41	Restrainer dimensions
Hasegawa et al. 1999 [36]	2	130	22	1291	rolled steel JIS G 3101 SS400	Global load-displacement response
Iwata et al. 2001 [14]	4	176	16	1296	rolled steel SN400B	Unbonded material
Nakamura et al. 2000 [37]	11	100	16–25	960–1180	LYP100(4), SN400B (3), LYP235(4)	Fatigue properties
Koetaka et al. 2001 [38]	7	120–150	22	1080–1910	SM490A with yield stress 345 N/mm^2 and SN490B with yield stress 364 N/mm^2	Safety factor against flexural buckling, ratio of sectional area at the end to that at the centre of braces, and width to thickness ratio at the end of braces.
Meritt et al. 2003 [39]	8	64–161	19-25	4470–4704	A36 steel, with a nominal yield strength, Fy= 248.2 MPa	Brace resultant force, brace axial deformation, hysteretic energy, cumulative inelastic axial deformation, compression strength adjustment factor, and tension strength adjustment factor
Black et al. 2004 [14]	5	145–204	19	3090–3410	JIS SM490A with yield stress 418.5 MPa and JIS SN400B with yield stress 285.4 MPa	Hysteretic behaviour
Iwata 2004 [40]	4	176	16	$2\sqrt{2}$ times story height	SN400B (Fy = 262.6 MPa and 289.1 MPa)	Hysteresis characteristics, final fracture characteristics, and cumulative absorbed energy
Tsai et al. 2004 [41]	10	100	20	900	A572 GR50	Unbonding material
Tremblay et al. 2006 [42]	2	125	12,7	1001–2483	G40 21-350WT steel with Fy = 370 MPa and Fu = 492 MPa	Brace axial load-core strain relationship
Ma et al. 2008 [43]	3	84	18	929	Q235 steel	Cruciform shape of core
Ding et al. 2009 [44]	10	70.8–100	7,5–7,64	1106–1789	Chinese Q235-B steel with nominal yield stress fy = 235 MPa and tensile stress fu = 420 MPa	Unbonded material, clearance between the panel and the brace, configuration of the steel bar and the edge reinforcement, and effective width of the panel
Eryasar 2009 [45]	12	100	10	900	European S355 grade (EN 10025, 1994) steel (Fy = 355 MPa and Fu = 510 MPa)	Global load-displacement response, adjustment factors, initial stiffness, yielding, and buckling patterns

n - number of specimens, b - width of core [mm], t - hickness of core [mm], L - yielding length of core [mm]

Table 1. Summary of experimental studies on BRBs - continuation

Reference \ Parameters	n	b	t	L	Core material	Investigated parameter
Ju et al. 2009 [46]	7	100×100×6×8		1900–2100	SS400 steel with yield strength of 240 MPa	Thickness of the external tube and unconstrained part of the core
Chou i Chen 2010 [47]	4	150	22	2880	ASTM Gr 50 steel	Restraining member size, and elimination of unbonding material
Mirtaheri et al. 2011 [48]	4	80	8	600–1300	steel with yield stress 297.5 MPa	Yielding core length, energy dissipation, and effective stiffness
Fahenstock et al. 2012 [49]	2	35 6 i 38 1	12,7	1364	A36 steel with yield stress 305 MPa and 365 MPa	Hysteretic response and cumulative deformation capacities
Takeuchi et al. 2012 [50]	6	94–130	16–22	1000	steel core plate with yield stress 257-261 N/mm^2	Perpendicular force of the core plate, local buckling wave of the core plate, and ultimate strength of the restrainer wall
Hikino et al. 2013 [51]	2	74	12	30 and 110	HS63S-T5 aluminium alloy (206.3 MPa Yield stress)	Out-of-plane stability of BRB
Wang et al. 2013 [52]	10	100	10	1360	HS63S-T5 aluminium alloy (206.3 MPa Yield stress)	Extruded core portion
Zsarnoczay A 2013 [53]	10	40–55	15–20	1800–2000	S235 JR grade steel	Cumulative inelastic deformation capacity, material overstrength factor, strain hardening adjustment factor, compression strength adjustment factor, and total hardening adjustment under compression
Tabatabaei et al. 2014 [54]	2	80	10	1100	ST 37-2 (DIN 17100) with nominal yield strength of 235 MPa and ultimate strength of 365 MPa	Hysteretic responses
Zhao et al. 2014 [55]	8	49.5–54	5–8	1400	Q235-B	Core width-to-thickness (b/t) ratio and the gap between the core and casing
Chen et al. 2016 [56]	7	80–100	10	880–900	Q235B steel	Width of the in-plane gap and core dimension
Sahoo i Ghowsi 2017 [57]	2	40	8	1000	Fe410 with specified yield stress of 250 MPa	Hysteretic response, energy dissipation response, and displacement ductility
Jia et al. 2018 [58]	5	70	10	670	mild steel SS400 with a yield stress of 275.6 GPa	Initial stiffness, maximum ductility indices, cumulative ductility indices, and equivalent viscous damping ratios

n - number of specimens, b - width of core [mm], t - hickness of core [mm], L - yielding length of core [mm]

Table 1. Summary of experimental studies on BRBs - continuation

Reference \ Parameters	n	b	t	L	Core material	Investigated parameter
Qu et al. 2018 [59]	7					Fuse design, fuse material, debonding material, and loading protocol
Wang et al. 2018 [60]	6	100–150	10	1800–2100	Q195(5) with Yield stress 216 to 220 MPa and Q235B(1) with Yield stress 270 MPa to 280 MPa	Hysteresis curve, skeleton curve, tension and compression nonuniform coefficient, energy dissipation coefficient, equivalent viscous damping ratio, plastic deformation performance, and in-plane and out-of-plane lateral displacements of the external restraining plate of the low yield BRB
Li et al. 2019 [61]	6	20–24	6	230	Q235B with yield stress 251.3 MPa	Hysteresis response
Quang et al. 2019 [62]	2	60	20	1700	Q235 steel (nominal yield strength of 235 MPa)	Stiffness and bearing capacity of the specimens
Qu et al. 2020 [63]	6	56	10	500	Q235B steel made in China (235 MPa)	Strain-rate, compression overstrength, and cumulative deformation capacity of BRBs
Zhou et al. 2021 [64]	3	Miura-ori pattern with length 90 mm and height 69.28 mm	6	1472–1536	Q235 low-carbon steel	Hysteretic behaviour

n - number of specimens, b - width of core [mm], t - hickness of core [mm], L - yielding length of core [mm]

5.2. Numerical studies on BRBs

In the last few decades, a number of research studies have been conducted to determine the effectiveness and performance of BRBs. Clark et al. [65] in their study demonstrated that the frames equipped in BRBs may have larger residual drifts owing to the low post yielding stiffness of the brace. Similar results were shown by Sabelli et al. [15]. Fahenstock et al. [66] in his numerical simulation, evaluated the maximum ductility demand value which was 26 under six ground motions scaled to be 1.5 times larger than the design level intensity. They computed the value of the cumulative brace ductility demand to be 99 under the design earthquake intensity and 171 under the maximum expected intensity of the tremor. It was expected that the seismic design of these braces cannot be administered by a low-cycle fatigue mechanism. Kiggins and Uang [67] and Ariyaratana and Fahnestock [68] conducted studies on dual systems consisting of buckling restrained braced frames and moment resisting frames. They worked on the methods to reduce the residual drifts. The application of these braces in tall buildings was studied by Kim et al. [69]. They obtained good results in terms of the strength, stiffness, and ductile response of the braces.

A number of finite element analyses were performed on BRBs with different configurations and material properties. Fahenstock et al. [17] explained the nonlinear dynamic analyses that were conducted on BRB frames using ground motion records scaled to two seismic hazard levels. Takeuchi et al. [35] conducted nonlinear analyses to explain the local buckling mechanism of the outer tube. They deduced that when there is a larger gap between the core, the outer tube is adopted, and the thickness of the tube is relatively smaller, there is a significant increase in the strain rate of the tube. From their analyses, they also inferred that there is no effect of the length of the core member on the performance of the brace. Korzekwa and Tremblay [70] also carried out nonlinear analyses by applying cyclic loading on all-steel BRBs. They analysed the nature of the contact forces that developed between the core member and outer tube. They deduced that these forces were resisted by the bolts in tension and by the tube in flexure. These forces resulted in the development of longitudinal frictional forces that gave rise to compressive loads acting axially in the outer tube while the displacement cycles were imposed to the brace. Dusicka and Tinker [71] studied ultra-light weighted BRBs with a core made up of aluminium and bundled glass fibre-reinforced polymer pultruded tubes as restrainers. These BRBs were found to be effective against the global buckling stability mode and weighed approximately 27 % of the conventional filler type BRBs and approximately 41 % of the all-steel BRBs. Anniello et al. [72] theoretically evaluated the performance of all-steel dismountable BRBs using finite element analysis. The aim was to upgrade the existing reinforced concrete buildings that were already tested experimentally in the past. Hoveidae and Rafezy [73] carried out finite element analyses in all-steel BRBs to study the local buckling behaviour of a steel core plate. Karimi et al. [74] conducted finite element analysis on a three-story steel frame incorporating BRBs and evaluated the seismic response of the frame under an impact load and dynamic analysis. Rossi [75] numerically investigated these braces for the isotropic hardening rule. Hosseinzadeh and Mohebi [76] investigated the finite element models of all-steel BRBs under cyclic analysis and compared the performance of these braces with ordinary braces. They also examined the responses of the frames with all-steel BRBs for non-linear static and dynamic analyses. Almeida et al. [77] presented a case study by retrofitting an existing RC school building incorporating all-steel BRBs. They evaluated the behaviour of the building by non-linear pushover analysis and concluded that BRBs can also be used to strengthen existing buildings. Lin et al. [78] numerically investigated a damped-outrigger system incorporating BRBs as energy dissipating devices. The seismic performance of a single BRB outrigger system was evaluated using non-linear response-history analysis and found to be satisfactory for minor earthquakes. Rahnavard et al. [79] proposed a method to accurately model and construct a simple model of a BRB. They considered a case study on two specimens that were experimentally tested to model BRBs using finite element modelling with ABAQUS software. Avci-Karatas et al. [80] developed finite element models of earlier tested specimens using full-scale experimental data [81]. They modelled two steel cores with a steel restrainer and one aluminium alloy core with an aluminium restrainer and identified the key issues governing the hysteretic behaviour in BRBs. Alborzi et al. [82] proposed a hybrid BRB consisting of a core member made using multiple plates with different stress–strain behaviours. The behaviour of this innovative hybrid BRB was compared with that of conventional BRBs on three building frames with different heights using time-history analysis and it was deduced that the proposed hybrid BRB provided better energy dissipation. Jamkhaneh et al. [83] proposed a new type of all-steel BRB with corrugated edges of the core and external sheath and examined them using finite element modelling. It was observed that the corrugated and ribbed edges enhanced the buckling resistance of the braces. Naghavi et al. [84] numerically investigated different types of concentrically braced frames and BRB frames through non-linear pushover analysis and time-history analysis using ABAQUS. They observed that the BRBs undergo significant plasticity without forming plastic hinges, and thus dissipate a comparatively larger amount of energy. The BRB elements were found to delay yielding in the building frame.

5.3. Design criteria of BRBs

Currently, the frames equipped with all-steel BRBs are extensively applied in various countries including the US owing to their considerably better seismic performance and effectiveness as lateral load resisting structures. However, these frames have not been looked into in the 2005 edition of Seismic Provisions for Structural Steel Buildings by the American Institute of Steel Construction (AISC). However, AISC in collaboration with the Structural Engineer's Association of California have given certain design guidelines for the construction of such frames. These guidelines were made with an intention for inclusion in the 2005 edition of the abovementioned provisions [76]. The recommended provisions were thereafter reviewed and included in FEMA 450. The acceptance of the BRB theory for application in any plan requires that the brace should fulfil the criteria of section 8.6.3.7.10 of the 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450) [85]. The design of BRBs is considered to be based on the equivalent lateral force method in the US. It is important to note that the ultimate axial displacement is considered to be two times the axial displacement of the brace in the 2005 edition of the AISC Seismic Provisions for Structural Steel Buildings, whereas it is 1.5 times the axial displacement of the brace published in FEMA 450.

5.3.1. Seismic design inclusions

US design codes provide a response modification coefficient R for the seismic design of buildings and frames. This coefficient is used to minimize the elastic seismic forces, thereby presenting the nonlinear response of the structure. In European codes, the behaviour factor q is a similar factor [86]. However, BRB frames are not included. According to the "recommended provisions for buckling-restrained braced frames" published by the Structural Engineers Association of Northern California steel subcommittee of seismology, a response modification coefficient of 8 is suitable for non-moment-resistant beam-column connections and it should be 9 for moment-resistant beam-column connections. However, these values were reduced to 7 and 8, respectively, when submitted to the "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures" [87]. These values were then adopted by the ASCE/SEI 7 standard "Minimum design loads for buildings and other structures" [88]. Comparing the response modification coefficient as given by the American codes with the behaviour factor by Eurocode 8, it can be deduced that the value of q for a highly ductile moment resisting frame will be approximately 6.5 to 8, which is considerably less than the R value of 7 to 8 for BRB frames given by US standards. Alternatively, the value of q for concentrically braced frames is taken as 4.8 in Eurocode 8. The design of BRB frames is often governed by ultimate limit state design methods. Moreover, these frames have a comparatively higher stiffness and hence, the value of q will be greater. This implies that the use of frames equipped with BRBs would be economical in European seismic areas than other types of frames. Moreover, the repairing costs in other types of frames is considerably higher than that of BRB frames because BRBs can be replaced easily after any seismic event. The application of all-steel BRBs is therefore preferred than the conventional unbonded type of BRBs because they are easy to fabricate, install, and replace.

Mahamoudi and Zaree [89] also attempted to evaluate the *R* value for BRB frames. They performed static nonlinear analysis on building models with single- and double-bracing bays, multi-floors, and different brace configurations. They obtained high values of the *R* factor for BRB frames. They also observed that the *R* value is highly affected by the building height and the number of bracing bays.

Moni et al. [90] also conducted a study to determine the *R* value for low to medium rise BRB frames according to the National Building Code of Canada (NBCC) 2010. The maximum *R* value was found to be 6.17 for the BRB frame. It was also observed by the researchers that the frames with greater heights had lower *R* values.

Abou-Elfath et al. [91] from their research proposed a new response modification factor for BRB frames to be considered in Egyptian codes. There is no defined *R* value for BRB frames in Egyptian codes and hence the frames with BRBs are designed using an *R* value of 4.5, which is used in the conventional bracing system. They deduced that the *R* value for BRB frames is considerably higher.

5.3.2. Core size

The design of the steel core size is quite simple if the required axial yield strength is known, which can be calculated easily. According to the AISC 2005 provisions [32], for a BRB frame, the moment resistance from the beams and columns are not considered when determining the required yield strengths of the BRBs. Hence, a braced bay acts as a statically determinant truss and can be analysed easily for design. The cross-section area of the core of the brace Ac, can be calculated as expressed in equation (20).

$$A_c = \frac{P_y}{\phi F_y} \tag{20}$$

where P_y yield strength required, F_y yield strength of the steel core, and ϕ the strength reduction factor which is equal to 0.9 for BRBs.

The materials specified as per the AISC guidelines for the steel core includes JIS G 3136 SN400 B, ASTM A36, or ASTM A572 Grade 42.

5.3.3. Casing design

The selection of the casing material and size design is not as simple as that of the core. The casing part restrains the inner core from buckling and hence, it has to be very carefully designed. The design of the casing primarily depends on the maximum axial force that will be transferred by the inner steel core. This force depends on the frictional force between the core and casing, material overstrength factor of the steel core, restrained lateral deformation of the steel core, and material hardening. This force may also be larger than the full plastic capacity of the casing cross-section. According to the AISC guidelines [32], the maximum force transferred by the brace is as given in equation (21),

$$P_{max} = \beta \cdot \omega \cdot R_y \cdot P_y \tag{21}$$

where β is the compression strength adjustment factor, which is equal to the ratio of the maximum compressive and maximum tensile force; ω is the strain hardening adjustment factor, which is equal to the ratio of the maximum tensile strength and design yield strength; and R_y is the material overstrength factor, which is equal to the ratio of the maximum base shear in the actual behavior to the first significant yield strength in the structure.

For the casing or restrainer, the materials specified by the AISC guidelines include JIS G3466 STKR 400, or ASTM A500, Grade B. However, other casing materials can also be used if qualified by testing. Some researchers have proposed other materials other than steel that can be used as the casing material. Rahai et al. [92] used PVC pipes and FRP sheets as the outer covering material and investigated such BRBs experimentally and numerically and concluded that these materials were suitable alternatives for steel. Partial BRBs were also proposed by Abraham [93]; they were made of fibre-reinforced polymer-stabilized steel members through a retrofit application.

5.3.4. Other elements

Other elements include the unbonding material (if applied), type of connection, and brace connection part. For all-steel BRBs, the main application function is not to coat the core with unbonding material but it is to restrict the bonding between the core and restrainer. Alternatively, a small gap can be provided between the casing and core. For all-steel BRBs, the casing portion is attached with the core. This could be done by either welding or bolting. For the design of the bolts, Wu [94] obtained a formula for calculating the maximum tensile load demand N_s on the bolts in equation (22).

$$N_s = \frac{4S_s + 2\gamma\varepsilon_c w_c}{L_c} P_{max} \tag{22}$$

where S_s gap between core and restrainers on the strong side, γ plastic Poisson's ratio, ε_c expected maximum core strain, w_c width of the all-steel BRB core, L_c Centre-to-centre distance between the two bolts, and P_{max} maximum load carried by the all-steel BRB at the ultimate level.

The portion of the core that extends beyond the casing is connected to the frame by gusset plates. These extruding portions are non-yielding and can be connected to the frame through bolted connections. Currently, pinned connections are preferred over standard and modified bolted connections because of their lower installation cost and negligible overturning moment. They also facilitate the use of longer core length BRBs, which result in a lower strain.

The guidelines available do not consider the extruded portion of the core outside the casing. This portion may suffer torsional buckling or may fail before the yielding portion of the core and hence requires more attention.

5.3.5. Alternatives

Apart from the various codal recommendations from different countries, different approaches for the design of BRBs that are installed in the moment resisting frames have been studied. The force-based method has always been the first choice of in earlier studies; however, currently, the displacement- and energy-based methods are gaining attention in the design of BRB frames. Performance-based design using the energy-based method is becoming popular in the performance of BRB frames during seismic events. Moreover, all-steel BRBs are also gaining popularity among designers for application in BRB frames because they are easy to fabricate as well as replace or repair after any seismic event.

Housner [95] was the first researcher to come up with the concept of the energy-based design method. Kim et al. [96] proposed a seismic design method for steel frames with BRBs based on the energy balance concept. They computed the hysteretic demand of the steel frame and accordingly designed the size of the BRBs. Ye et al. [97] presented a design framework for the seismic design of steel structures with braced frames. Avila et al. [98] proposed a method for the energy-based seismic design of frames according to the performance-based earthquake engineering method. Ma [99] proposed an energy-based method for the seismic design of steel eccentrically braced frames. The method was based on the hysteretic energy spectra and accumulated ductility ratio spectra. It is believed that the design of BRB frames is to some extent similar to that of eccentrically braced frames, and hence, the proposed method can be used for the seismic design of BRB frames. However, the method was proposed according to the Chinese soil classification.

5.4. Failure modes to be observed on BRBs

The following failure modes should be observed while carrying out analysis on BRBs in either numerical or experimental studies.

5.4.1. Global buckling of the assembly

The most important stability governing criteria of BRBs is to check it against the overall buckling of the brace; this needs to be given prior consideration. Majority of studies conducted in the past focused on this failure mode only. The researchers proposed different limiting force conditions to prevent global buckling failure. Earlier, it was proposed that no global buckling takes place when the Euler buckling load of the restrainer is greater than the yielding stress of the inner core even if the core member is subjected to an intense compressive force. Further, different values of the ratio of the Euler buckling load of the restraining member to the yield strength of the core, P_e/P_y were proposed for design purposes. Hoveidae and Rafezy [100] proposed this ratio to be 1.2.

5.4.2. Local buckling of the inner core

Takeuchi et al. [101] examined the local buckling behaviour of the core member and evaluated the effects of the outer tube thickness on the buckling behaviour of these braces. They also justified that there is no use of studying the behaviour of BRBs in the elastic range because these braces suffer large inelastic deformations owing to strong ground motions. Hoveidae and Rafezy [73] found that the friction coefficient between the core and restrainer, the gap between them, and the configuration of the interface have significant effects on the local buckling behaviour of the inner steel core. They suggested that an appropriate gap size with an unbonding agent of appropriate thickness having a smaller friction coefficient, can prevent local buckling of the steel core by providing free lateral expansion of the core. They also suggested that the application of unbonding material was a better option than the direct contact option and the gap option.

5.4.3. Necking of the inner core

Failure due to the necking of the core is encountered generally when the number of braced frames is low and the BRBs used are short; short BRBs are normally stiffer. This develops large forces on the BRBs that are few in number. As a result, the inner core of the BRB inhibits excessive stresses at high tensile forces. These stresses further reach the ultimate strength of the material and consequently, the strain give rise to the necking of the inner core. Although the behaviour of the core after yielding is stable and ductile, the reversal of the load from necking negotiates the BRB structural behaviour. Therefore, the concern of the occurrence of such situations should be addressed at the design stage.

5.4.4. Buckling of inner core post-necking

When the necking of the inner core occurs because of the reversal of the load, it remains in its initial stage. This necking generates weak points in the steel core, and the core fails in the uniform distribution of strain along its yield length when the load reversal is in compression. Therefore, the weak points that are formed damage the core by developing uneven buckling. The uneven buckling of the core produces a transverse force on the restraining unit in which the core is being encased. Subsequently, bulging of the encasement takes place.

5.4.5. Out-of-plane core buckling

The portion of the inner core that extends outside the restraining unit can buckle during the reversal of load during compression. Manufacturers should therefore stiffen these end portions against weak-axis buckling. This could be achieved by perpendicular stiffener plates or stabilizing collars. It was proposed that when the embedment length is approximately 1.5 to 2 times greater than the width of the core at the yielding portion, no loss of the flexural stiffness occurs along the length of the core [102]. Furthermore, the parallel orientation is preferred because it is unfavourable for the out-of-plane buckling condition. Figure 6 shows some typical failure modes in BRBs.

5.5. Applications of BRBs

BRBs have practical applications in a number of projects worldwide, especially in Japan. The earlier BRBs installed in buildings were unbonded with filler materials; however, all-steel BRBs are currently preferred. All-steel BRBs are simple to install in the frames because of their light weight and easy fabrication. The installation of BRBs in steel bridges as seismic devices has proven to be the most efficient method of damage control. BRBs in steel bridges are expected to efficiently resist major earthquakes three times without being replaced [103].

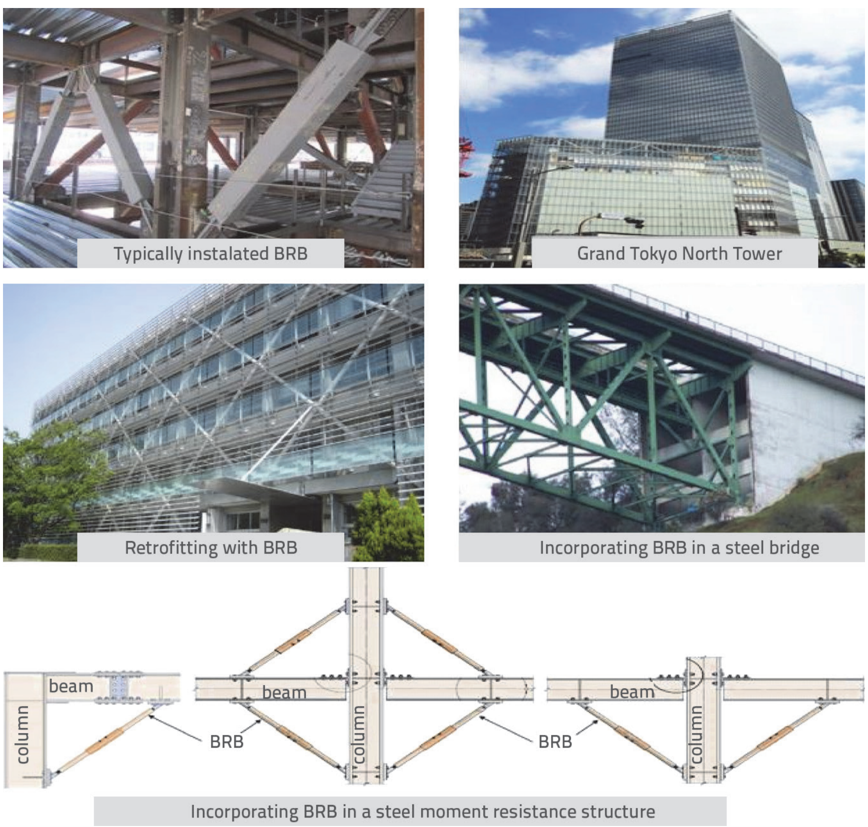

Figure 7. Applications of BRBs [1, 103, 104]

The use of BRBs as dampers in steel moment resisting systems can attain satisfactory performance. BRBs can be used for the strengthening of plate girders in bridge constructions as shown in Figure 7. Large span bridges can achieve efficient retrofitting with BRBs owing to their better architectural flexibility and ease of installation [104]. BRBs are applied in various other countries including the US, China, Taiwan, Turkey, and New Zealand. Despite being widely used in America and Japan, the BRB system lacks a standardized design approach in the Eurocodes. According to the European EN 15129 standard [105] on anti-seismic devices, BRBs are categorized as nonlinear displacement dependent devices. Directly applying the US design criteria cannot be a practical technique because there are substantial differences between the European and US approaches to structural design regulations. Zsarnóczay [106] thus proposed design procedures for the BRB frame design conforming to the Eurocode, by testing BRB specimens provided by Star Seismic Europe Ltd.
Several retrofit projects have selected BRBs for dissipating energy and improving the seismic behaviour of the existing buildings in Taiwan. A ten-story gymnasium building was constructed at the Chinese Culture University in Taipei using BRBs as energy dissipating devices. In non-ductile moment frames, BRBs are mostly installed in the perimeter as an external frame. Such types of retrofits are generally difficult to apply. For these situations, "integrated facade engineering" is being proposed [107]. This concept combines the structural retrofit, facade design, and environmental design, and includes improvements on the seismic performance using seismic energy dissipation devices as BRBs. BRBs are also applied in trusses and spatial structures. The Toyota stadium is an example of the application of BRBs to the supporting structure of a spatial structure. Moreover, BRBs have been applied in the construction of a number of bridges in recent years. Recently, BRBs have been used in a rocking or spine frame commonly identified as the "strong-back system".

6. Conclusion

In this paper, authors summarized the concept, development, stability criteria, experimental and numerical research, design considerations, failure modes to be observed, and applications of buckling restrained braces (BRBs) over the past decades. It can be concluded that BRBs are very effective seismic devices that have proven their usefulness. Moreover, they are economical compared with other seismic devices. With regards to budget construction or low-to-medium rise building constructions, BRBs are the best alternative for application as seismic devices. Moreover, all-steel BRBs are preferred to the conventional BRBs owing to their low cost, less weight, and easy installation. The frames equipped with BRBs have a comparatively higher stiffness and larger energy dissipation than the moment-resisting and concentrically-braced frames. The authors attempted to review the studies carried out on BRBs and presented a brief summary on these braces depending on the availability of literature and past research. Overall, it can be concluded that BRBs are a recent and economical solution to earthquake-resistant building designs; they are effective and reliable seismic devices.

The global buckling failure of braces is the predominant stability criteria for which BRBs are designed. Additional research is required in this area. Further, stability on the connection portion and local buckling also require additional experimental research. BRBs have a low post-yield stiffness as compared to the initial yield stiffness, and this may result in damage on one level; it is necessary to find solutions to this problem. Very few researchers have attempted to use materials other than steel as the outer restrainer member. According to the authors, this could be an important modification of the braces as it would result in cost saving and light-weight construction. Furthermore, hollow steel sections can be used as restrainers instead of heavy steel sections. Such sections are lighter with a desirable moment of inertia; this aspect requires additional practical research. Another aspect that has not been given full consideration is the confinement effect between the core member and surrounding filler material. This aspect needs to be experimentally and numerically studied. The tendency of BRB frames to have larger residual displacements may be considered as a limitation; however, this is also a property of a one elastic-plastic device. Today, numerous countries are conducting research on BRBs and applying them practically; however, their efforts are focused on developing a testing system to which patented systems can be produced by companies. Consequently, these braces are not used in countries where the demand is low. Moreover, to the best of the authors' knowledge, these braces have not been included in the design codes of most countries but have been used in various earthquake prone countries repeatedly. Therefore, it is necessary to formulate simpler and better guidelines for their design. However, there are many aspects that are still unaddressed and need to be studied in this field. This may be the reason why most countries have still not included BRBs in their seismic design code. With the growth of technology, in the recent construction and design of buildings, the need for energy dissipating technology has also increased, and BRBs have proven their effectiveness as energy dissipating devices. The future of BRBs is very bright in seismic construction; thus, additional experimental and numerical studies are required so that designers can gain more confidence in their application.

Acknowledgements

The manuscript was developed at the Visvesvaraya National Institute of Technology, Nagpur, India. The authors are grateful to the institute for the support provided to the first author as part of her doctoral research.

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